Terraprobe
Consulting Geotechnical & Environmental Engineering
Construction Materials Inspection & Testing

GEOTECHNICAL INVESTIGATION
PROPOSED MIXED USE
COMMERCIAL AND RESIDENTIAL DEVELOPMENT
COLLINGWOOD SHIPYARDS – BLOCK 6
COLLINGWOOD, ONTARIO

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1.0 INTRODUCTION

Terraprobe Inc. (Terraprobe) was retained by FRAMSlokker to conduct a geotechnical investigation for a proposed residential/commercial development located on the north side of Side Launch Way, approximately 100 m west of Heritage Drive and adjacent to Collingwood Harbour. The property is identified as Collingwood Shipyard Block 6, Town of Collingwood, Ontario.

Terraprobe carried out a preliminary geotechnical investigation of the larger Collingwood Shipyards Area in 2003 that included the advancement of four boreholes on the subject site. The findings of this previous investigation are documented in our report titled “Preliminary Geotechnical Design CSL Lands, Collingwood, Ontario” File No. 1-03-0290, dated October 6, 2003.

This report encompasses the results of the geotechnical investigation conducted for the proposed mixed use residential/commercial development to determine the prevailing subsurface soil and groundwater conditions, and based on this information, provides geotechnical design recommendations for the foundations, basement floor slab, basement drainage, pavement, and earth pressure and seismic design parameters. Geotechnical comments are also included on pertinent construction aspects, excavation, backfill and groundwater control, and shore retaining walls (anchored steel sheet piles pinned to bedrock).

2.0 SITE AND PROJECT DESCRIPTIONS

The site is located on the north side of Side Launch Way, approximately 100 m west of Heritage Drive and adjacent to Collingwood Harbour. The property is identified as Collingwood Shipyard Block 6, Town of Collingwood, Ontario. The general location of the site is presented on Figure 1. **At the start of the investigation (Nov. 2020), the water level on Georgian Bay (CHS Collingwood Gauge) was approximately at Elev. 177.20 m. The site was for the most part covered by water as the surface topography is at or below this elevation.**

The site consists of an irregular shaped parcel of land and is currently vacant.

Based on the drawing sets titled “Collingwood Block 6, Preliminary Design & Statistics” dated August 26, 2019 “Collingwood Block 6 Design Update” dated November 23, 2020, prepared by Giannone Petricone Associates, it is understood that the proposed development would include a six (6) storey residential building, with a retail space to the south and supported over one level of underground parking (P1). The project will also include at grade parking and driveways supported over the parking deck. The proposed P1 finished floor level will be set at Elev. 174.20 m to 175.20 m. It should be noted that the scope for the geotechnical investigation was based on the Preliminary Design & Statistics drawings which was provided at the time that the proposal was prepared.

The planned project will also include shore retaining walls (anchored steel sheet pile walls pinned to bedrock).

3.0 INVESTIGATION PROCEDURE

The field investigation was conducted was conducted on November 5, 2020 and March 3 and 4, 2021, and consisted of drilling and sampling a total of eleven (11) boreholes, extending to about 3.1 m to 8.2 m depth below grade. Moreover, the investigation was conducted in two stages. Eight boreholes (Boreholes 1 to 8) extending to a depth of 3.1 to 3.9 m below grade were carried out on November 5, 2020. Subsequently, seven rock probes (Boreholes 101, 103, 105, 106, 107, 109 and 110) and three sampled boreholes (Boreholes 102, 104 and 108), including the extraction of 3 m of rock core from each borehole, and extending to depths ranging from 2.4 to 8.2 m below grade were carried out on March 3 and 4, 2021 to investigate the subsurface soil and bedrock conditions for the proposed shore retaining walls. The approximate locations of the boreholes are shown on the enclosed Borehole Location Plan (Figure 2A – Existing Condition and Figure 2B and 2C – Proposed Condition). The boreholes logs for boreholes carried out in November 2020 and March 2021 are provided in Appendix A. The pertinent borehole logs from the 2003 investigation are provided in Appendix B, for information purposes.

The boreholes were drilled by a specialist drilling contractor using track-mounted drill rig power auger. The borings were advanced through the overburden soil to the bedrock surface using continuous flight solid stem augers, and were sampled at 0.75 m intervals (up to 3.0 m depth) and 1.5 m intervals (below 3.0 m depth) with a conventional 50 mm diameter split barrel sampler when the Standard Penetration Test (SPT) was carried out (ASTM D1586). Coring was advanced by NQ core at Boreholes 102, 104 and 108. As noted earlier, Boreholes 101, 103, 105, 106, 107, 109 and 110 were carried out as rock probes, therefore the boreholes were advanced directly to the inferred bedrock surface without sampling. The field work (drilling, sampling and testing) was observed and recorded by a member of our field engineering staff, who logged the borings and examined the samples as they were obtained.

All samples obtained during the investigation were sealed into clean plastic jars, and transported to our geotechnical testing laboratory for detailed inspection and testing. All borehole samples were examined (tactile) in detail by a geotechnical engineer, and classified according to visual and index properties. Laboratory tests consisted of water content determination on all samples; and Sieve and Hydrometer analysis on five (5) selected native soil samples (Borehole 1, Sample 3; Borehole 3, Sample 4B, Borehole 8, Sample 4, Borehole 102, Sample 5 and Borehole 108, Sample 3). The measured natural water contents of individual samples and the results of the Sieve and Hydrometer analysis are plotted on the enclosed Borehole Logs at respective sampling depths. The results of Sieve and Hydrometer analysis are also summarized in Section 4.6 of this report, and appended.

Water levels were measured in open boreholes upon completion of drilling. Standpipe piezometers comprising 25 mm diameter PVC pipes were installed in selected boreholes (Boreholes 1 and 7) to facilitate groundwater monitoring. The PVC tubing was fitted with a bentonite clay seal as shown on the accompanying Borehole Logs. Water levels in the piezometers were measured on November 16, 2020.

The results of groundwater monitoring are presented in Section 4.7 of this report.

The borehole ground surface elevations were surveyed by Terraprobe using a Trimble R10 GNSS System. However, Borehole 5 could not be surveyed due to the large amount of water present on the ground surface at the time of the survey. The Trimble R10 system uses the Global Navigation Satellite System and the Can-Net reference system to determine target location and elevation. The Trimble R10 system is reported to have an accuracy of up to 10 mm horizontally and up to 30 mm vertically.

It should be noted that the elevations provided on the Borehole Logs are approximate only, for the purpose of relating soil stratigraphy and should not be used or relied on for other purposes.

4.0 SUBSURFACE CONDITIONS

The specific soil conditions encountered at each sampled borehole location are described in greater detail on the applicable Borehole Logs, with a summary of the general subsurface soil conditions outlined below. This summary is intended to correlate this data to assist in the interpretation of the subsurface conditions encountered at the site.

It should be noted that the subsurface conditions are confirmed at the borehole locations only, and may vary between and beyond the borehole locations. The boundaries between the various strata as shown on the logs are based on non-continuous sampling. These boundaries represent an inferred transition between the various strata, rather than a precise plane of geologic change.

4.1 Crusher run Limestone (Granular Fill)

A surficial layer of granular fill consisting of crusher run limestone was encountered in Boreholes 2, 3, 5, 6 and 8, and had a thickness varying from about 450 mm (Boreholes 3 and 5) to 750 mm (Boreholes 2, 6 and 8).

4.2 Earth Fill

A zone of earth fill material was encountered from the ground surface in Boreholes 1, 7, 102, 104 and 108 and beneath the crusher run limestone granular fill in Boreholes 2, 3, 5, 6 and 8, and extended to a depth varying from about 0.8 to 3.0 m below existing grade. The earth fill materials predominately consisted of silty sand (sand and gravel in Borehole 3) with inclusions of clay, gravel, and occasional cobble size particles. Sporadic and a trace amounts of organics, as well as presence of rootlets were noted within the fill materials at varying depths, and peat inclusions were noted at 1.5 m depth in Borehole 6. Wood fragments were identified in the fill in Borehole 102 and concrete fragments to trace concrete was evident in Boreholes 104 and 108, respectively.

The Standard Penetration Test results (‘N’ Values) obtained from the earth fill materials varied from 5 to 43 blows per 300 mm of penetration, indicating a typically loose to dense relative density. It should be noted that some of the relative high ‘N’ Values obtained from the earth fill materials may due to the presence of harder particles such as cobbles and may not necessarily represent the actual state of compactness of the material tested.

Measured moisture contents of the earth fill materials samples generally varied from about 9 to 30 percent by weight, indicating a generally wet condition.

At Boreholes 2 and 7, a layer of sand/silty sand was encountered below the fill and extended to a depth of 3.0 and 3.4 m below grade, respectively. There was no readily apparent structure to the material and this may be possible fill used to raise the site grades.

4.3 Silty Sand/Sandy Silt/Sand

A cohesionless deposit consisting of silty sand, sandy silt or sand was encountered below the fill in Boreholes 1, 2, 3 and 108, and extended to a depth varying from 2.4 to 3.7 m below grade, i.e. auger refusal depth in Borehole 2. The deposit contained inclusions of clay and gravel.

Standard Penetration Test ‘N’-values obtained from the cohesionless sand and silt deposit varied from about 28 to greater than 50 blows per 300 mm of penetration, indicating a compact to very dense (typically compact) relative density. The in- situ moisture contents of the sand and silt samples varied from 9 to 21 percent by weight, indicating a wet condition.

4.4 Sandy Silt to Silty Sand Till

A sandy silt to silty sand glacial till deposit, with trace to some gravel and trace clay was encountered beneath the cohesionless sand and silt deposit in Boreholes 1, 3 and 108 and below the fill layer in Boreholes 4 through 8, 102 and 104, and extended to depths ranging from 2.1 to 5.2 m below grade, i.e. the maximum depths explored/auger refusal, in Boreholes 1 to 8. At Borehole 5, the till is interrupted by a 400 mm thick sandy silt deposit which was encountered between a depth of 2.3 and 2.7 m below grade. Standard Penetration Test ‘N’-value for the layer was in the order of 29 blows per 300 mm of penetration, indicating a compact relative density and the in-situ moisture content was in the order of 29 percent by weight indicating a wet condition.

Standard Penetration Test ‘N’-values obtained from the sandy silt to silty sand till deposit varied from about 17 to greater than 50 blows per 300 mm of penetration, indicating a compact to very dense (typically very dense) relative density. The in-situ moisture contents of the glacial till samples varied from 5 to 22 percent by weight indicating locally wet condition.

4.5 Bedrock

The bedrock surface was inferred by auger/sampler refusal in Boreholes 1 to 8, 101, 103, 105, 106, 107, 109 and 110 and coring at Boreholes 102, 104, 108 and 03-10 (2003 Borehole) at depths varying from about 2.1 to 5.3 m below grade (Elev.171.4 to 174.5 ±m). The depth/elevation of the inferred or cored bedrock surface (including the rock probes carried out in Boreholes 101, 103, 105, 106, 107, 109 and 110) are provided in the following table.

Borehole	Ground Surface Elevation	Depth to top of Inferred Bedrock Surface	Elevation of Top of Inferred Bedrock Surface
1	177.1 m	3.9 m BG	173.2 m
2	177.0 m	3.7 m BG	173.3 m
3	176.7 m	3.3 m BG	173.4 m
4	176.8 m	3.1 m BG	173.7 m
5	176.3 m	3.8 m BG	172.5 m
6	176.6 m	3.6 m BG	173.0 m
7	176.7 m	3.5 m BG	173.2 m
8	176.7 m	3.8 m BG	172.9 m
101	176.3 m	2.7 m BG	173.6 m
102	176.5 m	3.7 m BG	172.8 m
103	176.7 m	3.0 m BG	173.7 m
104	176.0 m	2.1 m BG	173.9 m
105	176.5 m	2.9 m BG	173.6 m
106	176.5 m	3.2 m BG	173.3 m
107	176.7 m	2.4 m BG	174.3 m
108	176.7 m	5.3 m BG	171.4 m
109	176.9 m	3.0 m BG	173.9 m
110	176.8 m	3.4 m BG	173.4 m
03-10	178.1 m	3.6 m BG	174.5 m

BG – Below Existing Grade

Bedrock was confirmed by coring between 3.7 and 6.8 m (Elev. 172.8 and 169.7 m) depth below grade at Borehole 102, between 2.1 and 5.2 m (Elev. 173.9 and 170.8 m) depth below grade at Borehole 104 and between 4.7 and 7.7 m (Elev. 171.4 and 168.4 m) depth below grade at Borehole 108. Hence variability of the bedrock surface elevation is evident from the borehole findings.

The bedrock consists predominantly of limestone of the Simcoe Group. The rock is of medium strength and it is generally competent. RQD values ranged from 38 % to 62 %. An approximately 310 mm long fractured zone was observed near the surface of the bedrock in Borehole 102 and approximately 80 mm thick fractured zones were noted at 2.4 m and 4.1 m depths in Borehole 104 and at 7.2 m depth in Borehole 108. The bedrock is horizontally bedded.

4.6 Geotechnical Laboratory Test Results

The geotechnical laboratory testing consisted of natural water content determination for all samples, while Sieve and Hydrometer analysis were conducted on selected native soil samples. The test results are plotted on the enclosed Borehole Logs at respective sampling depths.

The results (graphs) of the Sieve and Hydrometer (grain size) analysis are appended and a summary of these results is presented as follows:

Borehole No. Sample No.	**Sampling Depth below Grade (m)**	**Percentage (by mass)**				**Descriptions (MIT System)**
		Gravel	**Sand**	**Silt**	**Clay**	
Borehole 1, Sample 3	1.8	24	37	30	9	SILTY SAND, gravelly, trace clay
Borehole 3, Sample 4B	2.6	10	35	50	5	SANDY SILT, some gravel, trace clay
Borehole 5, Sample 4	2.5	17	40	35	8	SILTY SAND, some gravel, trace clay
Borehole 102	3.3	9	40	49	2	SILT AND SAND, trace gravel, trace clay
Borehole 108	1.8	0	87	13	0	SAND, some silt

4.7 Groundwater

Observations pertaining to the depth of water level and caving were made in the open boreholes immediately after completion of drilling, and are noted on the enclosed Borehole Logs. Standpipe piezometers were installed in Boreholes 1 and 7 to facilitate shallow groundwater level monitoring. The groundwater level measurements in the piezometers were taken on November 16, 2020 and April 19, 2021 and are noted on the enclosed Borehole Logs. A summary of these observations is provided as follows:

Borehole No.	**Depth of Borehole (m)**	**Upon Completion of Drilling**		**Water Level in Piezometer on November 16, 2020**	**Water Level in Piezometer on April 19, 2021**
		Depth to Cave (m)	**Unstabilized Water Level (m)**	**Depth/Elev. (m)**	**Depth/Elev. (m)**
BH 1	3.9	Not measured	Not measured	-0.05*/177.05	0.15/176.95
BH 2	3.7	open	0.6	Piezometer not installed	Piezometer not installed
BH 3	3.3	Not measured	Not measured	Piezometer not installed	Piezometer not installed
BH 4	3.1	Not measured	Not measured	Piezometer not installed	Piezometer not installed
BH 5	3.8	Not measured	Not measured	Piezometer not installed	Piezometer not installed
BH 6	3.6	Not measured	Not measured	Piezometer not installed	Piezometer not installed
BH 7	3.5	Not measured	Not measured	-0.04*/176.74	0.04/176.66
BH 8	3.8	Not measured	Not measured	Piezometer not installed	Piezometer not installed
BH102	6.9	Not measured	Not measured	Piezometer not installed	Piezometer not installed
BH104	5.8	Not measured	Not measured	Piezometer not installed	Piezometer not installed
BH108	8.2	Not measured	Not measured	Piezometer not installed	Piezometer not installed

**Water level was recorded above the current ground surface within the stick-up of the piezometer. (ground surface is below the flood line)*

The site surface was covered by water at the time of investigation and the groundwater levels encountered within the piezometers (November 16, 2020 monitoring) was above the current ground surface, i.e. Elev. 177.05 m and 176.74 m at Boreholes 1 and 7, respectively. Water levels measured on April 19, 2021 indicated water levels at Elev. 176.95 m and 176.66 m in Boreholes 1 and 7, respectively. The site is adjacent to Georgian Bay which has a level that typically varies in the range of 176 to 177 metres geodetic elevation and was at Elev. 177.2 m on November 16, 2020. For design purposes, the stabilized groundwater table is at about Elev. 177.2 ±m.

5.0 DISCUSSIONS AND RECOMMENDATIONS

The following discussion and recommendations are based on the factual data obtained from this investigation and are intended for the use of the owner and the design engineer. Contractors bidding or providing services on this project should review the factual data and determine their own conclusions regarding construction methods and scheduling.

This report is provided on the basis of these terms of reference and on the assumption that the design features relevant to the geotechnical analyses will be in accordance with applicable codes, standards and guidelines of practice. If there are any changes to the site development features or there is any additional information relevant to the interpretations made of the subsurface information with respect to the geotechnical analyses or other recommendations, then Terraprobe should be retained to review the implications of these changes with respect to the contents of this report.

Considering the proposed P1 level (FFE at Elev. 174.20 m to 175.20 m) will be made in the wet cohesionless silty sand/sandy silt and glacial till below the stabilized groundwater table (Elev. 177.2 ±m), dewatering to a minimum of 1.0 m below the lowest excavation elevation must be carried out prior to any excavation and maintained at that level during construction. If the subsurface is not dewatered prior to excavation, the subgrade soils will become disturbed by the ingress of groundwater and the recommendations above for bearing capacity will not be valid.

Based on the preliminary drawing set with Project Title: “The Shipyards, Block 6, Shoreline Treatments Collingwood” prepared by Shoreplan Engineering Limited and dated 2021/02/19, it is understood that a shored wall system consisting of anchored steel sheet piles pinned to bedrock will encircle the site of the north, east and west (adjacent to water body). Based on the subsurface conditions and our experience from nearby sites, Terraprobe recommends constructing a continuous interlocking caisson wall shoring system, with the pile and filler toes advanced a minimum of 3 ±m into the bedrock to reduce the flow of groundwater into the excavation (i.e. create a groundwater cut-off). Dewatering inside the cut-off excavation may be conducted with conventional sumps or by other means and methods as determined by the professional dewatering contractor. The full height of the basement walls must be waterproofed and designed to withstand horizontal hydrostatic pressure below the stabilized groundwater table (Elev. 177.2 ±m). This arrangement will allow the proposed structures to be founded on conventional spread footings, with a subfloor drainage system to relieve uplift pressure.

The foundation installations must be reviewed in the field by Terraprobe. The on-site review of the condition of the foundation subgrade as the foundations are constructed is an integral part of the geotechnical engineering design function, and is not to be considered as third-party inspection services. If Terraprobe is not retained to carry out all of the foundation evaluations during construction, then Terraprobe accepts no responsibility for the performance of the foundations.

5.1 Foundation

Boreholes 1 and 2 are located within or in the vicinity of the retail space and Boreholes 3 to 8 are located within or in the vicinity of the 6-storey residential building footprint. The boreholes generally encountered crusher run limestone granular fill and/or cohesionless earth fill overlying cohesionless sand and silt and cohesionless sandy silt to silty sand glacial till, extending to the full depth of the investigation. Inferred bedrock surface was noted at about Elev. 172.9 to 173.7 ± m (depths varying from about 3.1 to 3.9 m below existing grade) in the boreholes due to auger/sample refusal.

It is understood that the proposed mixed-use development will include a six (6) storey residential building, with a retail space to the south, and supported over one level of underground parking (P1). The project will also include at grade parking and driveways supported over the parking deck. The proposed P1 finished floor level will be set at Elev. 174.20 m to 175.20 m. For design purposes, the stabilized groundwater table is at about Elev. 177.2 ±m, i.e. in line with the adjacent Georgian Bay.

Below the P1 level (FFE of Elev. 174. 20 m to 175.20 m, conventional spread footings can be made to bear on the undisturbed (dewatered) very dense native soils (sandy silt/sandy silt and silty sand to sandy silt glacial till). The following table summarizes the recommended geotechnical reaction and geotechnical resistance available at the borehole locations.

BH No.	**Proposed P1 Finished Floor Elevation (m)**	**Highest (Bottom) of Footing Depth below Existing Ground Surface (m)**	**Highest (Bottom) of Footing Elevation (m)**	**Max. Geotechnical Reaction at SLS (kPa)**	**Max. Factored Geotechnical Resistance at ULS (kPa)**	**Bearing Stratum**
1	174.20	3.3	173.8	500	750	Very Dense Sandy Silt to Silty Sand Till (wet)
2	174.20	3.3	173.7	500	750	Very Dense Sand (wet)
3	174.20	2.6	174.1	500	750	Very Dense Sandy Silt Till(wet)
4	174.20	2.6	174.2	500	750	Very Dense Sandy Silt Till (wet)
5	174.2	3.0	-	500	750	Very Dense Sandy Silt Till (wet)
6	174.20	3.3	173.3	500	750	Very Dense Sandy Silt Till (wet)
7	175.20	3.4	173.3	500	750	Very Dense Sandy Silt Till (wet)

8	175.20	3.3	173.4	500	750	Very Dense Silty Sand to Sandy Silt Till (wet)

Notes: ULS=Ultimate Limit States; and SLS=Serviceability Limit States

The design bearing pressures for footings supported on the very dense soils as recommended allow for up to 25 mm of total settlement. This settlement will occur as load is applied and is linear elastic and non-recoverable. Differential settlement is a function of spacing, loading and foundation size.

As noted earlier, shallow bedrock was inferred in all boreholes due to auger/sample refusal and selective rock coring was carried out in three boreholes advanced for the proposed shore wall system. As such, considerations can be given to, conventional spread footings made to bear on sound (unweathered) bedrock below about Elev. 172 to 173.0 ±m, and designed using a maximum factored geotechnical resistance at Ultimate Limit States (ULS) of 3,500 kPa. Serviceability Limit States (SLS) does not apply for shallow foundations bearing directly on bedrock since the loads required for appreciable settlement to occur would be much greater than the factored resistance at the ULS. Foundations installed in accordance with the above recommendations would be expected to experience very little settlement likely limited to the elastic deformation of the concrete.

Given the variability of the bedrock surface, the foundations may have to be made on bedrock as drilled piers (caissons). Where such caissons are less than 3 diameters in length they will be effectively drilled footings and can be designed for a maximum factored geotechnical resistance of up to 5000 kPa (ULS). If extended to greater depths, these units would behave as deep foundations and the maximum factored geotechnical resistance can be increased to 8000 kPa ULS.

If hand cleaned drilled piers are not possible then the units could be made to bear in frictional support by socketing the caissons into the top of the bedrock. Such units derive support only from the adhesion of the concrete cast into the rock socket. For preliminary design purposes, it can be assumed that a design adhesion of 1000 kPa is possible in the Simcoe Group. This value would have to be proven by a load test before it could be used for construction purposes. There is no experience to reference which allows us to predict load settlement performance for a unit in this rock formation.

Considering the proposed P1 finished floor level of Elev. 174.20 m to 175.20 m, it is expected that foundations will be made up to 2 to 3 ±m below the prevailing groundwater table at this site (Elev. 177.2 ±m). The groundwater table must be lowered a minimum of 1.0 m below the lowest excavation elevation prior to any excavation and maintained at that level during construction. If the subgrade soils are not dewatered prior to excavation and maintained throughout construction, the subgrade soils will become disturbed and the recommendations provided above for bearing capacity will not be valid.

It must be noted that seasonal fluctuations in the groundwater table may result in higher groundwater levels than observed and reported.

5.1.1 Foundation Installation

All exterior foundations and foundations in unheated areas must be provided with a minimum soil cover of 1.4 m or equivalent insulation for frost protection.

For foundations supported on soils, footings stepped from one level to another must be at a slope not exceeding 7 vertical to 10 horizontal.

Footings stepped from one level to another bearing on bedrock should be at a slope not exceeding 1 vertical to 1 horizontal for the above bearing pressures to be applicable. There must also be a minimum of 300 mm horizontal ledge between the edge of any footing and the top of the rock cut down to another footing.

It is recommended that all excavated footing base must be evaluated by a qualified geotechnical engineer to ensure that the founding soils exposed at the excavation base are consistent with the design bearing pressure intended by the geotechnical engineer.

It is noted that the native soils tend to weather rapidly and deteriorate on exposure to the atmosphere or surface water. Hence, foundation bases which remain open for an extended period of time should be protected by a skim coat of lean concrete.

Prior to pouring concrete for the footings, the footing subgrade must be cleaned of all deleterious materials such as softened, disturbed or caved materials, as well as any weathered rock or standing water. If construction proceeds during freezing weather conditions, adequate temporary frost protection for the footing bases and concrete must be provided. It should be noted that the bedrock surface can weather and deteriorate on exposure to the atmosphere or surface water; hence, foundation bases which remain open for an extended period of time should be protected by a skim coat of lean concrete.

5.2 Basement Floor Slab

It is understood that the P1 basement floor slab of the proposed structure will be set at about Elev. 174.20 m to 175.20 m, i.e. a depth of about 2.0 to 3.0 m below existing grade. As such, the P1 slab will be made on (dewatered) compact to very dense silty sand/sandy silt and silty sand to sandy silt glacial till or fill. The (dewatered) native soils constitute adequate subgrades for the support of a slab on grade. Where present, the fill must be removed and replaced with suitable soil free of organics and debris or imported Granular B (OPSS.MUMI 1010) placed in 150 mm thick lifts and each lift compacted to at least 98 percent Standard Proctor Maximum Dry Density (SPMDD). The modulus of subgrade reaction appropriate for the design of a fully drained slab-on-grade resting on the granular drainage layer overlying the very dense native soils is 25,000 kPa/m and 18,000 kPa/m for granular drainage layer overlying compacted fill.

Alternatively, in lieu of imported Granular B, considerations can be given to the use of 19 mm clear stone (OPSS.MUNI 1004) vibrated to a dense state. However, a a suitable non-woven geotextile filter (Terrafix 270R or equivalent approved by Terraprobe) must be installed (with a minimum 900 mm overlap) below the 19 mm clear stone; otherwise, without proper filtering there may be entry of fines from the surrounding subgrade soils into the clear stone layer. This loss of ground could result in a loss of support of the slab.

Prior to the construction of the slab, it is recommended that the glacial till subgrade be cut neat, or proof-rolled for the sandy silt/silty sand subgrade, and inspected under the supervision of Terraprobe for obvious loose or disturbed areas as exposed, or for areas containing excessively deleterious materials or moisture. The native soil subgrade should be assessed and approved by Terraprobe prior to the placement of the slab-on-grade. Any disturbed areas shall be recompacted in place and retested, or else replaced with Granular B placed as engineered fill (in lifts 150 mm thick or less and compacted to a minimum of 98 % SPMDD). A static drum roller should be used to proof-roll the soils at this site, as vibration may cause unwanted disturbance, dilation and/or reduction in strength of the silty sand/sandy silt.

It is necessary that building floor slabs be provided with a capillary moisture barrier and drainage layer. This is made by placing the slab on a minimum 200 mm layer of HL-8 Coarse Aggregate or 19 mm clear stone (OPSS.MUNI 1004) compacted by vibration to a dense state. The upper 50 mm of clear stone can be replaced with 50 mm of 19 mm crusher run limestone for a working surface. Provision of subfloor drainage is required in conjunction with perimeter drainage of the structure to collect and remove water that infiltrates under the floor, as discussed in Section 5.4.

Cohesionless soils will be encountered at the subgrade for the slab on grade. Therefore, a suitable non-woven geotextile filter (Terrafix 270R or equivalent approved by Terraprobe) must be installed (with a minimum 900 mm overlap) below the HL-8 Coarse Aggregate or 19 mm clear stone; otherwise, without proper filtering there may be entry of fines from the surrounding subgrade soils into the subfloor drainage layer. This loss of ground could result in a loss of support of the slab and clogging of the subfloor drainage system.

The water level must be lowered from inside the excavation to at least 1 m below the lowest excavation elevation prior to excavation for the duration of below grade construction. To preserve the integrity of the subgrade soils as approved by Terraprobe, a skim coat of lean concrete could be applied to create a trafficable surface.

Regardless of the approach to slab construction, the floor slabs that are to have bonded floor finishes (such as tiles with adhesives) should be provided with a capillary moisture/vapour barrier. The floor manufacturers have specific requirements for moisture/vapour barrier; therefore, the floor designer/architect must ensure that a provision of appropriate moisture/vapour barrier conforming to specific floor finish product requirements is incorporated in the project specifications. Adequate testing must be carried out to ensure acceptable levels of moisture/relative humidity in the concrete slab prior to the installation of floor finish. Studies indicate that a provision of 200 mm thick 19 mm clear stone base (OPSS MUNI 1004) under the slab provides a good capillary moisture barrier provided the granular base is positively drained. However, this provision does not replace the floor manufacturers' specific requirement(s) for a moisture/vapour barrier.

The under-slab vapour retarder specifications, selection and installation shall conform to ASTM E1745 and ASTM E1643. The moisture vapour measurement tests shall conform to RH: ASTM F2170, RH: ASTM F2420 and Calcium Chloride: ASTM F1869. The Surface Applied Moisture Vapour Barrier system shall meet the guidelines established in ASTM F3010-13.

5.3 Earth Pressure Design Parameters

Walls or bracings subject to unbalanced earth pressures must be designed to resist a pressure that can be calculated based on the following equation:

$$P = K [\gamma (h-h_w) + \gamma' h_w + q] + \gamma_w h_w$$

Where:

P = the horizontal pressure (kPa)

K = the earth pressure coefficient

h = the depth below the ground surface (m)

h_w = the depth below the groundwater level (m)

γ = the bulk unit weight of soil (kN/m^3)

γ_w = the bulk unit weight of water (9.8 kN/m^3)

γ' = the submerged unit weight of the exterior soil, ($\gamma_{sat} - \gamma_w$)

q = the complete surcharge loading (kPa)

Where the wall backfill can be drained effectively to eliminate hydrostatic pressures on the wall, this equation can be simplified to:

$$P = K[\gamma h + q]$$

This equation assumes that free-draining granular backfill is used and positive drainage is provided to ensure that there is no hydrostatic pressure acting in conjunction with the earth pressure. Resistance to sliding of retaining structures is developed by friction between the base of the footing and the soil. This friction **(R)** depends on the normal load on the soil contact (**N**) and the frictional resistance of the soil (**tan ϕ**) expressed as **R = N tan ϕ**. The factored geotechnical resistance at ULS is **0.8 R**.

Passive earth pressure resistance is generally not considered as a resisting force against sliding for conventional retaining structure design because a structure must deflect significantly to develop the full passive resistance.

The average values for use in the design of structures subject to unbalanced earth pressures at this site are tabulated as follow:

Parameter	**Definition**	**Units**
ϕ	angle of internal friction	degrees
γ	bulk unit weight of soil	kN/ m^3
K_a	active earth pressure coefficient (Rankine)	dimensionless
K_o	at-rest earth pressure coefficient (Rankine)	dimensionless
K_p	passive earth pressure coefficient (Rankine)	dimensionless

Stratum/Parameter	γ (kN/m^3)	Φ (degree)	K_a	K_o	K_p
Compacted Granular Fill Granular ‘B’ (OPSS.MUNI 1010)	21	32	0.31	0.47	3.25
Existing Earth Fill	19.0	28	0.36	0.53	2.77
Silty Sand/Sandy silt/Sand	21.0	32	0.31	0.47	3.25
Silty San to Sandy Silt Till	21.0	34	0.28	0.44	3.54
Limestone Bedrock	24.0	28	n/a	n/a	n/a

The above values of the earth pressure coefficients are for the horizontal backfill grade behind the wall. The earth pressure coefficients for inclined grade will vary based on the inclination of the retained ground surface.

5.4 Basement Drainage

To assist in maintaining dry basements and preventing seepage, it is recommended that exterior grades around the building be sloped away at a 2 percent gradient or more, for a distance of at least 1.2 metres.

A waterproofed foundation wall and drained base approach is recommended at this site. It is recommended that the subfloor drainage system consists of minimum 100 mm diameter perforated pipes wrapped in filter fabric spaced at a maximum spacing of 3 metres on centre. The pipes must be surrounded by a minimum of 200 mm of 19 mm clear stone/HL-8 Coarse Aggregate, and the pipe inverts should be a minimum 300 mm below the base of the slab. The elevator pits can be drained separately with an independent lower pumping sump or can be designed as water proof structures which are below the drainage level. A typical basement subdrain detail is provided in Appendix D. The subfloor drains can be constructed in trenches as shown in the typical detail, or alternatively they can be constructed on a flat subgrade subexcavated at least 300 mm below the base of the slab. The subdrain system should outlet to a suitable discharge point under gravity flow, or connected to a sump located in the lowest level of the basement. The water from the sump must be pumped out to a suitable discharge point/positive outlet. The installation of the drains as well as the outlet must conform to the applicable plumbing code requirements.

The subfloor drainage system is a critical structural element, since it keeps water pressure from acting on the basement floor slab. As such, the sump that ensures the performance of this system must have a duplexed pump arrangement for 100% pumping redundancy and these pumps must be on emergency power. The size of the pump should be adequate to accommodate the anticipated groundwater seepage and storm event flows.

5.5 Earthquake Design Parameters

The current Ontario Building Code stipulates the methodology for earthquake design analysis, as set out in Subsection 4.1.8.7. The determination of the type of analysis is predicated on the importance of the structure, the spectral response acceleration and the site classification.

The parameters for determination of Site Classification for Seismic Site Response are set out in Table 4.1.8.4.A. of the Ontario Building Code. The classification is based on the determination of the average shear wave velocity in the top 30 metres of the site stratigraphy, where shear wave velocity (v_s) measurements have been taken. Alternatively, the classification is estimated on the basis of rational analysis of undrained shear strength (s_u) or penetration resistance (N-values).

$$\upsilon_{s-avg} = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{\upsilon_{si}}} \qquad S_{u-avg} = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{s_{ui}}} \qquad N_{avg} = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{N_i}}$$

Shear Wave Velocity

Undrained Shear Strength

SPT N-values

Based on the borehole data (advanced to a maximum of 3.9 m depth below grade), it is understood that the proposed building will be founded on the silty sand/sandy silt or sandy silt/silty sand till deposit of compact to very dense relative density or bedrock. It is expected that the deeper stratigraphy in this area is at least as competent as the lowest proven strata in the boreholes. On this basis, preliminary site seismic classification may be taken as Site Class C for foundations in soil or Site Class B for structures founded on bedrock, according to Table 4.1.8.4.A of the Ontario Building Code. Tables 4.1.8.4.B. and 4.1.8.4.C. of the current Ontario Building Code provide the applicable acceleration and velocity based site coefficients. The applicable acceleration and velocity based site coefficients for Site Class B and C are provided as follows:

Site Class	**Values of F_a (acceleration based coefficients)**				
	$S_a(0.2) \le 0.25$	**$S_a(0.2) = 0.50$**	**$S_a(0.2) = 0.75$**	**$S_a(0.2) = 1.00$**	**$S_a(0.2) \ge 1.25$**
B	0.8	0.8	0.9	1.0	1.0
C	1.0	1.0	1.0	1.0	1.0

Site Class	**Values of F_v (velocity based coefficients)**				
	$S_a(1.0) \le 0.1$	**$S_a(1.0) = 0.2$**	**$S_a(1.0) = 0.3$**	**$S_a(1.0) = 0.4$**	**$S_a(1.0) \ge 0.5$**
B	0.6	0.7	0.7	0.8	0.8
C	1.0	1.0	1.0	1.0	1.0

5.6 Pavement Above Underground Parking Structure

It is understood that the project will include at-grade driveway and parking lot supported over the P1 below grade parking deck. The following design is provided for the pavement above the parking structure.

5.6.1 Typical Pavement Above Parking Garage

A typical Pavement Components make-up above a concrete deck slab are as follows:

-Reinforced Concrete Slab Substrate

-Waterproofing with Protection Board

-Drainage Board/Drainage Layer

-Filter Fabric

-Min 300 mm Base Granular ‘A’ or 19 mm Crusher run limestone

-65 mm HL8

-40 mm HL3

The parking garage concrete structure must be designed to support the applicable loads of vehicles, i.e. fire trucks, garbage trucks, etc. which will access the laneway. Compaction requirements to meet those provided in Section 5.6.3.

5.6.2 Drainage

Drainage measures shall be provided for the pavement above the garage deck. This is typically provided by provision of 2 to 3 % grade on the garage deck and installing a subdrain system. A perimeter drain at the edge(s) of the garage deck should be considered in addition to above noted subdrain.

5.6.3 General Pavement Recommendations

HL 3 and HL 8 hot mix asphalt mixes should be designed, produced and placed in conformance with OPSS 1150 and OPSS 310 requirements and pertinent Town's standards.

Granular subbase material should meet the requirements of OPSS.MUNI 1010 and Town's standards. Granular materials should be compacted to 100 percent SPMDD at ±2 percent of the OMC.

PG 58-28, conforming to OPSS.MUNI 1101 is recommended in the HMA surface and binder courses. Tack coat SS-1 should be applied between hot mix asphalt binder course and surface course.

5.7 Excavations

The boreholes data indicate that the earth fill materials (to depths varying from 0.8 to 3.0 m below ground surface) and undisturbed native soils extending to inferred bedrock at depths varying from 3.1 to 3.9 m below grade, would be encountered in the excavations. Excavations must be carried out in accordance with the *Occupational Health and Safety Act and Regulations for Construction Projects*. These regulations designate four (4) broad classifications of soils to stipulate appropriate measures for excavation safety.

The earth fill materials and native soils encountered in the boreholes are classified as Type 3 Soil above and Type 4 Soil below the prevailing groundwater level, under these regulations.

Where workmen must enter excavations advanced deeper than 1.2 m, the trench walls should be suitably sloped and/or braced in accordance with the Occupational Health and Safety Act and Regulations for Construction Projects. The regulation stipulates the steepest slopes of excavation by soil type as follows:

Soil Type	Base of Slope	Steepest Slope Inclination
1	within 1.2 metres of bottom of trench	1 horizontal to 1 vertical
2	within 1.2 metres of bottom of trench	1 horizontal to 1 vertical
3	from bottom of trench	1 horizontal to 1 vertical
4	from bottom of trench	3 horizontal to 1 vertical

Minimum support system requirements for steeper excavations are stipulated in the Occupational Health and Safety Act and Regulations for Construction Projects, and include provisions for timbering, shoring and moveable trench boxes.

The overburden soils can be removed by conventional excavation equipment. It should be noted that the glacial till deposit may contain larger particles (cobbles and boulders) that are not specifically identified in the Borehole Logs. The size and distribution of such obstructions cannot be predicted with borings, because the borehole sampler size is insufficient to secure representative samples of the particles of this size. Provision should be made in excavation contracts to allocate risks associated with time spent and equipment utilized to remove or penetrate such obstructions when encountered.

Under the Act and Regulations, bedrock of the Simcoe Group Formation is not considered a soil. Where the excavation penetrates the bedrock, a vertical excavation made in sound bedrock is nominally self-supporting provided the rock bedding is horizontally oriented. The vertical rock face must be inspected by a geotechnical engineer to ensure no other support system is required to prevent the spalling of loose rock, and to ensure that all loose material at risk of falling upon a worker is removed (Section 233 of the above noted regulations). Should it be deemed necessary, rock bolts can anchor a layer of protective mesh that will protect workers from loose material spalling from the face of excavation.

The rock excavation may be conducted through line drilling in conjunction with hoe ramming. The line drilling is a process of drilling closely-spaced vertical holes into the rock (100 mm diameter, 300 mm centre to centre spacing) to provide a preferential break path for the excavation in the vertical plane and allow much easier breaking of the bedrock with a hydraulic ram. Excavating detailed shapes for foundations and the edges of the excavation are normally accomplished with hoe mounted hydraulic rams. The ability to remove the rock in a vertical face without over-excavation and dislodging of additional rock is largely dependent on the skill of the machine operator.

Very hard limestone layer may be encountered during excavation. Provision must be made in excavation contracts to allocate risks associated with the time spent and equipment utilized to remove or penetrate such obstructions. In the case of excess rock removal not intrinsic to the project requirements, the risk and responsibility for the excess rock removal under these circumstances, and the supply and placement of the extra concrete to restore the foundation grade, must be addressed in the contract documents for foundations, excavation, and shoring contractors.

Controlled blasting is not recommended due to the close proximity of the excavation area to the nearby existing structures.

5.8 Groundwater Control

For design purposes, the stabilized groundwater table at the site is Elev. 177.2 ±m. In general, the bulk excavation to the proposed P1 level (FFE of Elev. 174.20 m to 175.20 m) will extend approximately 2 to 3 m below the stabilized groundwater table (Elev. 177.2 ±m) at this site and excavations for foundations, elevator pits and sumps will extend even deeper below the stabilized groundwater table.

Within the zone of excavation, the glacial till is sufficiently cohesionless to be considered a moderate permeability material, and the silty sand/sandy silt and sand are moderate to high permeability materials, which will typically permit the free-flow of water when penetrated below the stabilized groundwater table.

Based on the subsurface conditions and our experience from nearby sites, Terraprobe recommends constructing a continuous interlocking caisson wall shoring system, with the pile and filler toes advanced a minimum of 3 ±m into the sound bedrock to reduce the flow of groundwater into the excavation (i.e. create a groundwater cut-off). Dewatering inside the cut-off excavation may be conducted with conventional sumps or by other means and methods as determined by the professional dewatering contractor. The full height of the basement walls must be waterproofed and designed to withstand horizontal hydrostatic pressure below the stabilized groundwater table (Elev. 177.2 ±m). This arrangement will allow the proposed structures to be founded on conventional spread footings, with a subfloor drainage system to relieve uplift pressure.

The dewatering system must remain on until such time as the subfloor drainage system and sumps are fully operational.

The relevant project information must be provided to a professional dewatering contractor who will be responsible for the design and installation of the dewatering systems. The dewatering system must be properly installed and screened to ensure that sediment and fine soils are not removed, which could result in settlement of the ground or structures near the site.

Flows from pumps due to ground water seepage and large rain storms during excavation and construction are expected to exceed 50,000 L/day.

5.9 Backfill

The earth fill materials containing excessive amounts of organic inclusion should not be reused as backfill in settlement sensitive areas, such as beneath the floor slabs, trench backfill and pavement areas. However, these materials may be stockpiled and reused for landscaping purposes.

The existing earth fill materials are considered suitable (with selection and sorting as required) for backfill provided the moisture content of these soils is within ±2 percent of the OMC. Any soil material with ±2 percent or higher in-situ moisture content than its OMC, could be put aside to dry or be tilled to reduce the moisture content so that it can be effectively compacted. Alternatively, materials of higher moisture content could be wasted and be replaced with imported material which can be readily compacted.

The existing earth fill materials will likely require selection and sorting to be reused as backfill. The selection and sorting must be conducted under the supervision of a geotechnical engineer. The site soils will be best compacted with a heavy sheep foot type roller.

The backfill should consist of clean earth and be placed in lifts of 150mm thickness or less, and heavily compacted to a minimum of 95 percent SPMDD at a water content close to optimum (within 2 percent). The upper 600 mm of the pavement subgrade (at driveways outside of the basement roof deck) must be compacted to a minimum of 98 percent SPMDD.

It should be noted that the soils encountered on the site are generally not free draining, and will be difficult to handle and compact should they become wetter as a result of inclement weather or seepage. Hence, it can be expected that the earthworks will be difficult and may incur additional costs if carried out during the wet periods (i.e. spring and fall) of the year.

5.10 Shoring Design Consideration

The site is bounded by Side Launch Way to the south, and by Nottawasaga Bay (Georgian Bay) to the north, east and west. No excavation shall extend below the foundations of existing adjacent Shore structures without adequate alternative support being provided.

Where excavations cannot be sloped, they can be supported using a shoring system such as soldier piles and lagging shoring or a continuous interlocking caisson wall shoring (as recommended for this site). Continuous interlocking caisson wall shoring is to be used where the excavation must be constructed as a rigid shoring system, to preserve the integrity and support of the soil beneath existing foundations of the adjacent shore structures in a state approximating the at-rest condition or for groundwater cut-off, for the bulk excavation.

For ground water control, it is recommended that the entire excavation at this site be shored as a continuous interlocking caisson wall shoring system, with the caisson pile and filler toes advanced a minimum of 3 ±m into the bedrock to cut off the ground water table in the wet silty sands and silts and glacial till, precluding horizontal ground water flow in the excavation. For this situation, dewatering can likely be accomplished by pumping from inside the sealed excavation using methods as determined by the professional dewatering subcontractor. It is assumed that the side/foundation walls of the structure will be waterproofed and designed to withstand hydrostatic pressure, and that the base of the structure will be drained in conjunction with a permanent concrete cut-off wall (caisson wall) socketed into cut-off deposit (sound bedrock).

The shoring system would best be supported by pre-stressed soil anchors extending beneath the adjacent lands and municipal roads. Pre-stressed anchors are installed and stressed in advance of excavation and this limits movement of the shoring system as much as is practically possible. The use of anchors on adjacent properties requires the consent of the adjacent land owners, expressed in encroachment agreements. The Town's Transportation and Works Department negotiates “permits” for the encroachment in City lands, which are generally allowed.

5.10.1 Earth Pressure Distribution

If the shoring is supported with a single level of earth anchor or bracing, a triangular earth pressure distribution similar to that used for the basement wall design is appropriate, and defined by:

$$\boldsymbol{P = K[\gamma H + q]}$$

where, $\boldsymbol{P}$ = the horizontal pressure at depth, **H** (kPa)
$\boldsymbol{K}$ = the earth pressure coefficient
$\boldsymbol{H}$ = the total depth of the excavation (m)
γ = the bulk unit weight of soil, (kN/m3)
$\boldsymbol{q}$ = the complete surcharge loading (kPa)

Where multiple supports are used to support the excavation, research has shown that a distributed pressure diagram more realistically approximates the earth pressure on a shoring system of this type, when restrained by pre-tensioned anchors. The multi-level supported shoring supporting predominately cohesionless soil can be designed based on an earth pressure distribution consisting of a rectangular pressure distribution with a maximum pressure defined by:

$$\boldsymbol{P = 0.65\, K\, (\gamma H + q)}$$

where, $\boldsymbol{P}$ = the horizontal pressure at depth, **h** (kPa)
$\boldsymbol{K}$ = the earth pressure coefficient
$\boldsymbol{H}$ = the total depth of the excavation (m)
$\boldsymbol{\gamma}$ = the bulk unit weight of soil, (kN/m3)
$\boldsymbol{q}$ = the complete surcharge loading (kPa)

For groundwater pressure distribution along the shoring wall in conjunction with the above soil pressures, the stabilized groundwater table should be taken at Elev. 177.2 ±m. The groundwater pressure distribution is only applicable where an impermeable boundary condition is created along the perimeter of the excavation, as is the case with a continuous interlocking caisson wall. Conventional soldier pile and lagging do not experience the water pressures, as water is allowed to drain freely through the wall.

5.10.2 Caisson and Soldier Pile Toe Design

Caisson piles and fillers are recommended to extend through the sand and silt and glacial till and made to bear in the sound bedrock below Elev. 171 to 174 ±m. The factored ultimate vertical bearing capacity for the design of a pile, embedded in the sound bedrock, is 10 MPa. The factored ultimate lateral bearing capacity of the unweathered rock is at least 1 MPa. Rock pins can be designed for 620 kPa SLS in compression and 400 kPa for uplift.

The horizontal resistance of the solider pile toes will be developed by embedment below the base of the excavation, where resistance is developed from passive earth pressure. It is noted that the resistance from these soils will be different depending on whether the soils are dewatered, or remain below the groundwater level. Where soils exist beneath the groundwater level, the unit weight of the soil is diminished by buoyancy. The design of the shoring will therefore have to consider the construction plan and sequence with respect to depth of groundwater control.

The soils at this site are cohesionless, permeable and sufficiently wet such that augered borings made into these soils will be unstable. It is necessary to advance temporarily cased holes to prevent excess caving during the soldier pile and all augered hole installations. Drill holes for piles, caissons, and/or fillers, utilizing temporary liners, mud drilling techniques, and/or other methods as deemed necessary by the contractor may be required to prevent issues such as: groundwater inflow or loss of soil into the drill holes, and disturbance to placed concrete.

5.10.3 Shoring Support

If anchor support is necessary and determined to be feasible, the shoring system should be supported by pre-stressed soil anchors extending beneath the adjacent lands. Pre-stressed anchors are installed and stressed in advance of excavation and this limits movement of the shoring system as much as is practically possible. It will be necessary to secure encroachment agreements from the Region/Town and the adjacent land owners, in order to use soil anchors on the adjoining properties. Pre-construction condition surveys should be carried out for the adjacent structures to establish existing conditions prior to excavation and mitigate the possibility of spurious claims for excavation induced damages. Access to the properties for such surveys must be part of any encroachment agreements. A careful evaluation of the subsurface soil conditions is required by the shoring designer to establish appropriate levels/elevations and design the soil anchors. The anchor design will be governed by the weakest material in the profile. It is imperative that a detailed design is carried out at different anchor levels and locations, and the anchors must be tested at each level.

Conventional earth anchors could be made with continuous hollow stem augers or alternatively post-grouted anchors can be made. The design adhesion for earth anchors is controlled as much by the installation technique as the soil and therefore a proto-type anchor must be made in each anchor level executed to demonstrate the anchor capacity and validate the design assumptions. A proto-type anchor must be made to demonstrate the anchor capacity (performance tested to 200% of the design load). All production anchors must be proof-tested to 133% of the design load, to validate the design assumptions.

The subsurface soils are sufficiently cohesionless, permeable and/or wet that augered holes could experience caving. It will be necessary to advance temporarily cased holes to maintain sidewall support and to prevent the ingress of water during installation, use slurry, etc. or other means or methods deemed necessary by the contractor.

Conventional earth anchors made in the generally dense to very glacial till or sandy silt may be designed using a working adhesion of 40 to 50 kPa. It is expected that post-grouted anchors can be made such that an anchor will likely carry about 60 to 70 kN/m of adhered anchor length (at a nominal diameter of 150 mm) in the dense to very dense glacial tills and sandy silt depending upon the material type as confirmed by a performance/load test. It should be noted that these values are provided as preliminary guidance only and the actual anchor performance must be verified by a performance/load test.

Alternatively, rock anchors can be considered extending into the bedrock (Simcoe Group) and can be designed using a working adhesion of 620 kPa.

Given the proximity of the existing/proposed shore walls and caissons walls, soil anchors may conflict anchors from existing or proposed shore walls. As such, rock anchors may be best suited for the proposed project. Coordination between the shore wall and caisson wall designs will be imperative for this project.

Regardless, the subsurface soil information should be reviewed by the shoring designer to decide on the suitable type of earth anchors and anchor capacity to be employed at this site.

If adjacent land owners are not agreeable to anchored support or limitations are presented by the adjacent Nottawasaga Bay then internal bracing or rakers would be necessary. The dense to very dense sands and silts, and glacial till below the proposed P1 level (FFE of Elev. 174.20 m to 175.20 m) are suitable for the placement of raker foundations. Raker footings established on the undisturbed (dewatered) native soils at an inclination of 45 degrees can be designed for a maximum factored geotechnical resistance at ULS of 300 kPa.

It will be necessary to secure encroachment agreements from the Region/Town and the adjacent land owners, in order to use soil anchors on the adjoining properties. Pre-construction condition surveys should be carried out for the adjacent structures to establish existing conditions prior to excavation and mitigate the possibility of spurious claims for excavation induced damages. Access to the properties for such surveys must be part of any encroachment agreements.

A careful evaluation of the subsurface soil conditions is required by the shoring designer to establish appropriate levels/elevations and design the soil anchors. The anchor design will be governed by the weakest material in the profile. It is imperative that a detailed design is carried out at different anchor levels and locations, and the anchors must be tested at each level.

5.11 Quality Control

Excavations on this site must be shored to preserve the integrity of the surrounding properties and structures. The current Ontario Building Code stipulates that engineering review of the subsurface conditions is required on a continuous basis during the installation of earth retaining structures. Terraprobe should be retained to provide this review, which is an integral part of the geotechnical design function as it relates to the shoring design considerations. Terraprobe can provide detailed shoring design services for the project, if requested. All foundations must be monitored by the geotechnical engineer on a continuous basis as they are constructed. The on-site review of the condition of the foundation soil as the foundations are constructed is an integral part of the geotechnical design function and is required by Section 4.2.2.2 of the current Ontario Building Code. If Terraprobe is not retained to carry out foundation evaluations during construction, then Terraprobe accepts no responsibility for the performance or non-performance of the foundations, even if they are ostensibly constructed in accordance with the conceptual design advice provided in this report.

Concrete for this structure will be specified in accordance with the requirements of CAN3 - CSA A23.1. Terraprobe maintains a CSA certified concrete laboratory and can provide concrete sampling and testing services for the project as necessary.

The requirements for fill placement on this project should be stipulated relative to SPMDD, as determined by ASTM D698. In-situ determinations of density during fill placement by Procedure Method B of ASTM D2922 are recommended to demonstrate that the contractor is achieving the specified soil density. Terraprobe is a CNSC licensed operator of appropriate nuclear density gauges for this work and can provide sampling and testing services for the project as necessary.

Terraprobe can provide thorough in-house resources, quality control services for Building Envelope, Roofing and Structural Steel in accordance with CSA W178, as necessary, for the Structural and Architectural quality control requirements of the project. Terraprobe is certified by the Canadian Welding Bureau under W178.1-1996.

6.0 LIMITATIONS AND RISK

6.1 Procedures

This investigation has been carried out using investigation techniques and engineering analysis methods consistent with those ordinarily exercised by Terraprobe and other engineering practitioners, working under similar conditions and subject to the time, financial and physical constraints applicable to this project. The discussions and recommendations that have been presented are based on the factual data obtained by Terraprobe.

It must be recognized that there are special risks whenever engineering or related disciplines are applied to identify subsurface conditions. Even a comprehensive sampling and testing programme implemented in accordance with the most stringent level of care may fail to detect certain conditions. Terraprobe has assumed for the purposes of providing design parameters and advice, that the conditions that exist between sampling points are similar to those found at the sample locations. The conditions that Terraprobe has interpreted to exist between sampling points can differ from those that actually exist.

It may not be possible to drill a sufficient number of boreholes or sample and report them in a way that would provide all the subsurface information that could affect construction costs, techniques, equipment and scheduling. Contractors bidding on or undertaking work on the project should be directed to draw their own conclusions as to how the subsurface conditions may affect them, based on their own investigations and their own interpretations of the factual investigation results, cognizant of the risks implicit in the subsurface investigation activities so that they may draw their own conclusions as to how the subsurface conditions may affect them.

6.2 Changes in Site and Scope

It must also be recognized that the passage of time, natural occurrences, and direct or indirect human intervention at or near the site have the potential to alter subsurface conditions. Groundwater levels are particularly susceptible to seasonal fluctuations.

The discussion and recommendations are based on the factual data obtained from this investigation made at the site by Terraprobe and are intended for use by the owner and its retained designers in the design phase of the project. If there are changes to the project scope and development features, the interpretations made of the subsurface information, the geotechnical design parameters and comments relating to constructability issues and quality control may not be relevant or complete for the revised project. Terraprobe should be retained to review the implications of such changes with respect to the contents of this report.

This report was prepared for the express use of FRAMSlokker and their retained design consultants and is not for use by others. This report is copyright of Terraprobe Inc. and no part of this report may be reproduced by any means, in any form, without the prior written permission of Terraprobe Inc. and FRAMSlokker who are the authorized users.

It is recognized that the regulatory agencies in their capacities as the planning and building authorities under Provincial statues, will make use of and rely upon this report, cognizant of the limitations thereof, both expressed and implied.

We trust the foregoing information is sufficient for your present requirements. If you have any questions, or if we can be of further assistance, please do not hesitate to contact us.

Yours truly,

Terraprobe Inc.

LICENSED PROFESSIONAL ENGINEER
O.B.
O. O. BENJAMIN
90387072
Sp 24, 2021
PROVINCE OF ONTARIO

O. Benjamin

Osbert (Ozzie) Benjamin, P.Eng.
Senior Project Manager, Geotechnical

Michael Tanos

Michael Tanos, P. Eng.
Principal

FIGURES

TERRAPROBE INC.

REFERENCE
OpenStreetMap@ 2020 Microsoft

Terraprobe
11 Indell Lane, Brampton, Ontario, L6T 3Y3
Tel: (905) 796-2650 Fax: (905) 796-2250

Title:

SITE LOCATION PLAN

File No.
1-20-0118-01

FIGURE :
1

T:\1-Project Files\2020\Branch Folders\3-20-0117 - Collingwood Shipyards Block 6, Collingwood\01 - Geotechnical Investigation\B. Drawings, Logs, Locates, etc\AutoCAD\3-20-0117-01 FIG 1.dwg, Sandy

REFERENCE

Google Earth @ 2020 Google

LEGEND

● Approximate Borehole Location (Terraprobe 2021)
⊕ Approximate Borehole Location (Terraprobe 2020)
◎ Approximate Borehole Location (Terraprobe 2003)

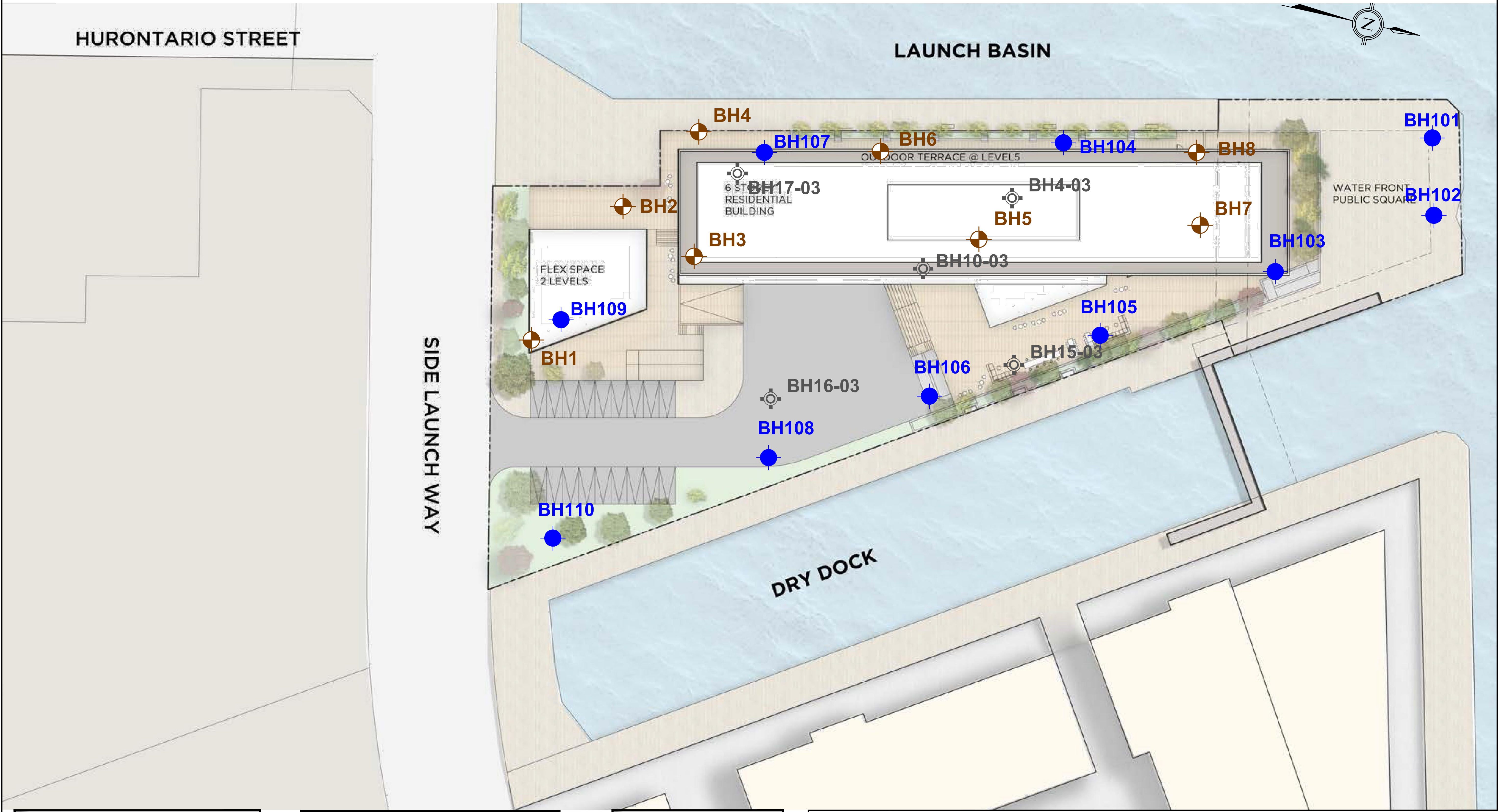
SCALE 1:600
0 2 4 6 8 10 20m

Terraprobe
11 Indell Lane, Brampton, Ontario, L6T 3Y3
Tel: (905) 796-2650 Fax: (905) 796-2250

Title:
BOREHOLE LOCATION AND SITE FEATURES PLAN
(EXISTING CONDITION)

File No. 3-20-0117

FIGURE : 2A



REFERENCE
Collingwood Block 6 Preliminary Design & Statistics
Site Plan Option 2, Date: 2019-08-26
By Giannone Petricone Associates

LEGEND
● Approximate Borehole Location (Terraprobe 2021)
⊕ Approximate Borehole Location (Terraprobe 2020)
◈ Approximate Borehole Location (Terraprobe 2003)

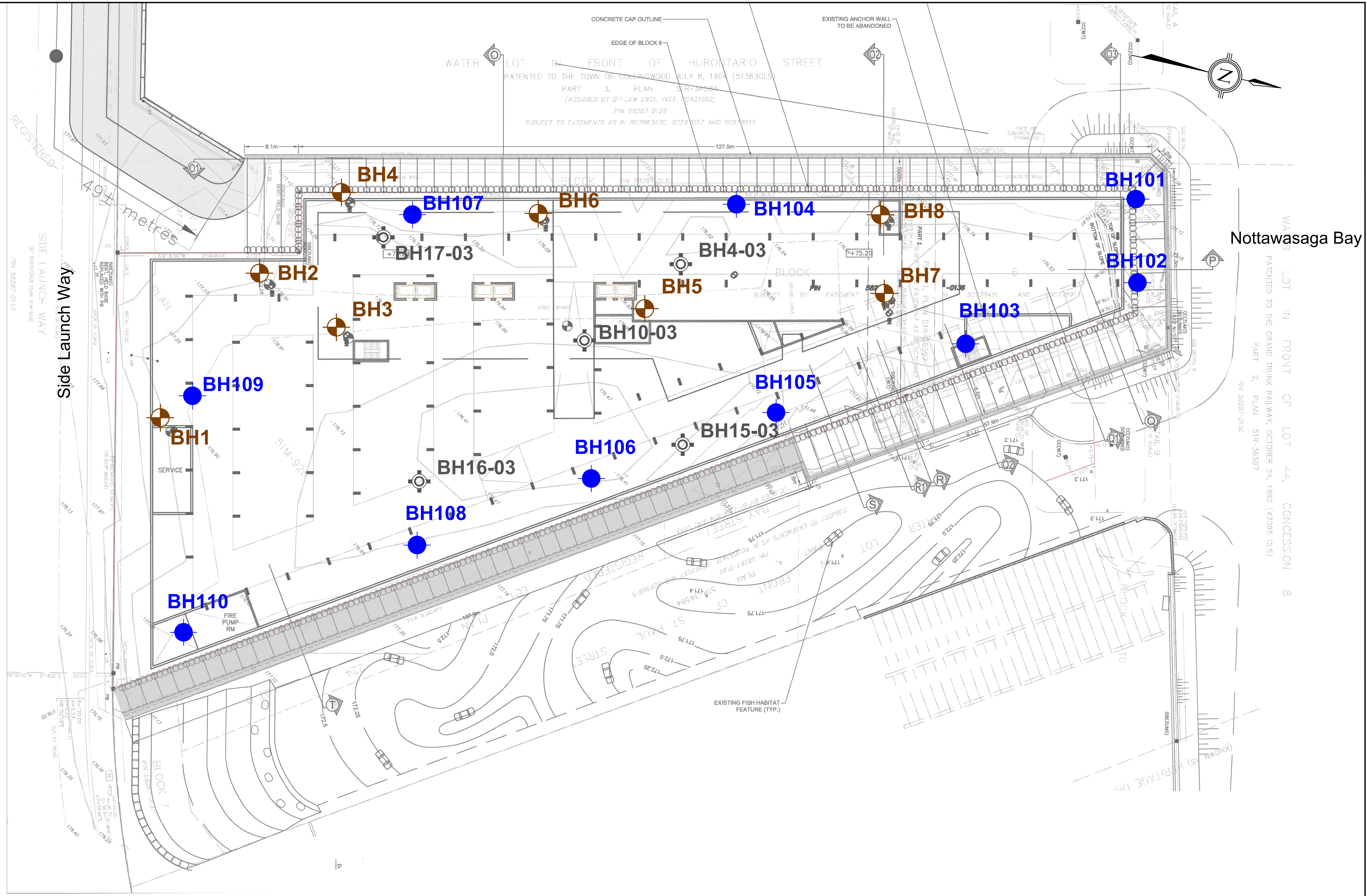
SCALE 1:600
0 2 4 6 8 10 20m

Terraprobe
11 Indell Lane, Brampton, Ontario, L6T 3Y3
Tel: (905) 796-2650 Fax: (905) 796-2250

Title:
BOREHOLE LOCATION AND SITE FEATURES PLAN
(PROPOSED CONDITION)

File No. 3-20-0117

FIGURE :
2B



REFERENCE
The Shipyards, Block 6 Shoreline
Treatments, Collingwood
Option 2, Site Plan
Project 17-2725, Sheet No. 01
By: Shoreplan Engineering Limited

LEGEND
- ● Approximate Borehole Location (Terraprobe 2021)
- ● Approximate Borehole Location (Terraprobe 2020)
- ◎ Approximate Borehole Location (Terraprobe 2003)

SCALE 1:600
0 2 4 6 8 10 20m

Terraprobe
11 Indell Lane, Brampton, Ontario, L6T 3Y3
Tel: (905) 796-2650 Fax: (905) 796-2250

Title:
BOREHOLE LOCATION AND SITE FEATURES PLAN
(PROPOSED CONDITION)

File No. 3-20-0117

FIGURE :
2C

APPENDICES

TERRAPROBE INC.

APPENDIX A

TERRAPROBE INC.

Terraprobe

ABBREVIATIONS AND TERMINOLOGY

SAMPLING METHODS

AS	auger sample
CORE	cored sample
DP	direct push
FV	field vane
GS	grab sample
SS	split spoon
ST	shelby tube
WS	wash sample

PENETRATION RESISTANCE

Standard Penetration Test (SPT) resistance ('N' values) is defined as the number of blows by a hammer weighing 63.6 kg (140 lb.) falling freely for a distance of 0.76 m (30 in.) required to advance a standard 50 mm (2 in.) diameter split spoon sampler for a distance of 0.3 m (12 in.).

Dynamic Cone Test (DCT) resistance is defined as the number of blows by a hammer weighing 63.6 kg (140 lb.) falling freely for a distance of 0.76 m (30 in.) required to advance a conical steel point of 50 mm (2 in.) diameter and with 60° sides on 'A' size drill rods for a distance of 0.3 m (12 in.)."

COHESIONLESS SOILS

Compactness	'N' value
very loose	< 4
loose	4 – 10
compact	10 – 30
dense	30 – 50
very dense	> 50

COHESIVE SOILS

Consistency	'N' value	Undrained Shear Strength (kPa)
very soft	< 2	< 12
soft	2 – 4	12 – 25
firm	4 – 8	25 – 50
stiff	8 – 15	50 – 100
very stiff	15 – 30	100 – 200
hard	> 30	> 200

COMPOSITION

Term (e.g)	% by weight
trace silt	< 10
some silt	10 – 20
silty	20 – 35
sand *and* silt	> 35

TESTS AND SYMBOLS

MH	mechanical sieve and hydrometer analysis
w, w_c	water content
w_L, LL	liquid limit
w_P, PL	plastic limit
I_P, PI	plasticity index
k	coefficient of permeability
γ	soil unit weight, bulk
G_s	specific gravity
φ'	internal friction angle
c'	effective cohesion
c_u	undrained shear strength

▽	Unstabilized water level
⧨	1st water level measurement
⧩	2nd water level measurement
▼	Most recent water level measurement
3.0 +	Undrained shear strength from field vane (with sensitivity)
C_c	compression index
c_v	coefficient of consolidation
m_v	coefficient of compressibility
e	void ratio

FIELD MOISTURE DESCRIPTIONS

Damp	refers to a soil sample that does not exhibit any observable pore water from field/hand inspection.
Moist	refers to a soil sample that exhibits evidence of existing pore water (e.g. sample feels cool, cohesive soil is at plastic limit) but does not have visible pore water
Wet	refers to a soil sample that has visible pore water

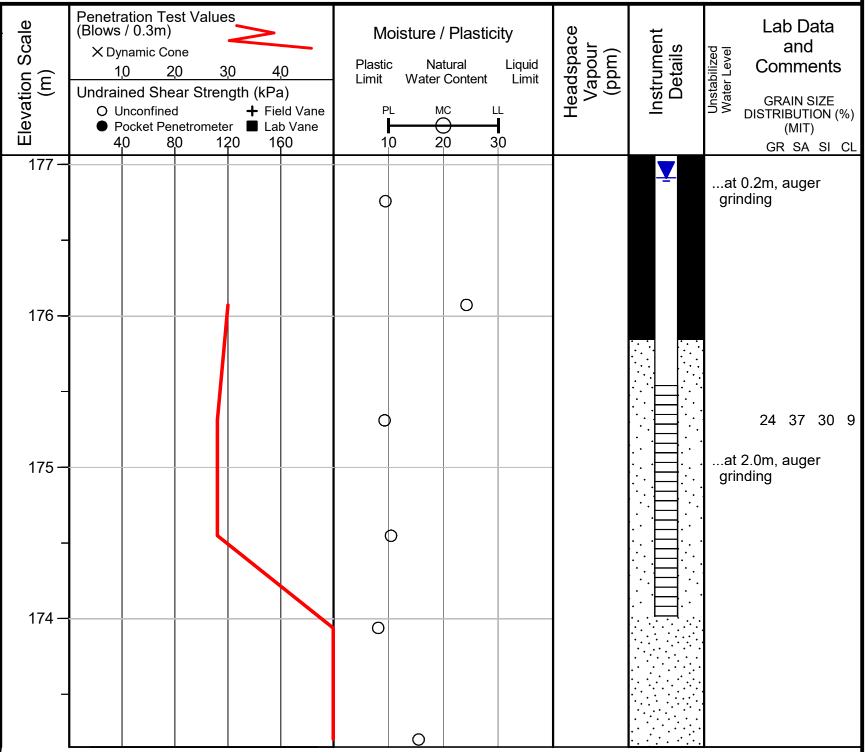
Terraprobe

LOG OF BOREHOLE 1

Project No. : 3-20-0117-01
Client : F.S. Collingwood Development Ltd.
Originated by : RS
Date started : November 5, 2020
Project : Collingwood Shipyards Block 6
Compiled by : HS
Sheet No. : 1 of 1
Location : Collingwood, ON
Checked by : OB

Position : E: 562251, N: 4928141 (UTM 17T)
Elevation Datum : Geodetic
Rig type : Track-mounted
Drilling Method : Solid stem augers

Depth Scale (m)	SOIL PROFILE		SAMPLES			Lab Data and Comments				
	Elev Depth (m)	Description	Number	Type	SPT 'N' Value	GRAIN SIZE DISTRIBUTION (%) (MIT) GR	SA	SI	CL	Comments
	177.1	**GROUND SURFACE**								
0		**FILL**, silty sand, some gravel, trace clay, trace rootlets, brown, wet	1	AS						...at 0.2m, auger grinding
1			2	SS	30					
	175.6 1.5	**SILTY SAND**, gravelly, trace clay, compact, grey, wet	3	SS	28	24	37	30	9	
2										...at 2.0m, auger grinding
			4	SS	28					
		...at 2.7 m, gravels								
3	174.1 3.0	**SANDY SILT to SILTY SAND**, trace clay, trace gravel, very dense, grey, wet (GLACIAL TILL)	5	SS	50 / 150mm					
	173.2 3.9		6	SS	50 / 100mm					



END OF BOREHOLE
Auger refusal on inferred bedrock

Water level in the well is 0.05m above the ground surface on November 16, 2020.

Water level was not measured due to unstabilized water depth.

25 mm dia. piezometer installed.

WATER LEVEL READINGS

Date	Water Depth (m)	Elevation (m)
Apr 19, 2021	0.2	176.9

file: 3-20-0117 bh logs_corr bh elev.gpj

Terraprobe

LOG OF BOREHOLE 2

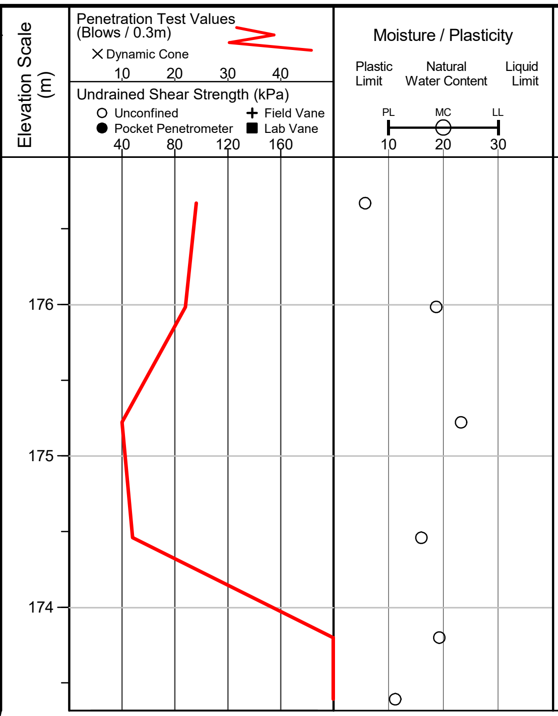
Project No. : 3-20-0117-01
Client : F.S. Collingwood Development Ltd.
Originated by : RS
Date started : November 5, 2020
Project : Collingwood Shipyards Block 6
Compiled by : HS
Sheet No. : 1 of 1
Location : Collingwood, ON
Checked by : OB

Position : E: 562227, N: 4928152 (UTM 17T)
Elevation Datum : Geodetic
Rig type : Track-mounted
Drilling Method : Solid stem augers

SOIL PROFILE		SAMPLES		
Elev Depth (m)	Description	Number	Type	SPT 'N' Value
177.0	**GROUND SURFACE**			
	750mm **CRUSHER RUN LIMESTONE**	1	SS	24
176.2 0.8	**FILL**, silty sand, trace to some gravel, trace clay, trace organics, loose to compact, grey, moist to wet	2	SS	22
	...at 1.5 m, wet below	3	SS	10
174.7 2.3	**SAND**, trace silt, trace gravel, compact, grey, wet (POSSIBLE FILL)	4	SS	12
174.0 3.0	**SAND**, some silt, trace gravel, trace clay, grey, wet	5	SS	50 / 100mm
		6	AS	50 / 150mm
173.3 3.7				

END OF BOREHOLE
Auger refusal on possible bedrock

Unstabilized water level measured at 0.6 m below ground surface; borehole was open upon completion of drilling.



Depth Scale (m): 0, 1, 2, 3

Headspace Vapour (ppm)

Instrument Details

Unstabilized Water Level: ∇

Lab Data and Comments
GRAIN SIZE DISTRIBUTION (%) (MIT)
GR SA SI CL

Terraprobe

LOG OF BOREHOLE 3

Project No. : 3-20-0117-01
Date started : November 5, 2020
Sheet No. : 1 of 1

Client : F.S. Collingwood Development Ltd.
Project : Collingwood Shipyards Block 6
Location : Collingwood, ON

Originated by : RS
Compiled by : HS
Checked by : OB

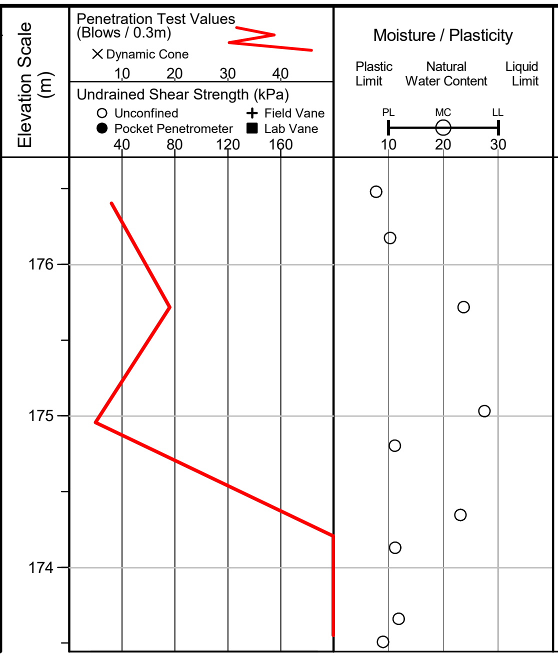
Position : E: 562233, N: 4928165 (UTM 17T)
Rig type : Track-mounted

Elevation Datum : Geodetic
Drilling Method : Solid stem augers

Depth Scale (m)	Elev Depth (m)	SOIL PROFILE Description	SAMPLES Number	Type	SPT 'N' Value	Lab Data and Comments GRAIN SIZE DISTRIBUTION (%) (MIT) GR SA SI CL
0	**176.7**	**GROUND SURFACE**				
		450mm **CRUSHER RUN LIMESTONE**, loose, white, moist	1A	SS	8	
	176.2 0.5	**FILL**, sand and gravel, trace silt, loose, brown, moist	1B			
1		**SILTY SAND**, trace to some gravel, trace clay, grey, wet	2	SS	19	
			3A	SS	5	
2	174.9 1.8	**SANDY SILT**, trace to some gravel, trace to some clay, loose, grey, wet	3B			
	174.4 2.3	**SILTY SAND**, trace to some gravel, trace clay, very dense, grey, wet	4A	SS	91 / 275mm	
	174.3 2.4	**SANDY SILT**, trace gravel, trace clay, very dense, grey, wet (GLACIAL TILL)	4B			10 35 50 5
3		...at 3.1 m, silty sand, some gravel	5A 5B	SS	50 / 65mm	
	173.4 3.3					

END OF BOREHOLE
Auger refusal on possible bedrock

Water level was not measured due to unstabilized water depth.



Headspace Vapour (ppm)

Instrument Details

Unstabilized Water Level

LOG OF BOREHOLE 4

Project No. : 3-20-0117-01
Client : F.S. Collingwood Development Ltd.
Originated by : RS

Date started : November 5, 2020
Project : Collingwood Shipyards Block 6
Compiled by : HS

Sheet No. : 1 of 1
Location : Collingwood, ON
Checked by : OB

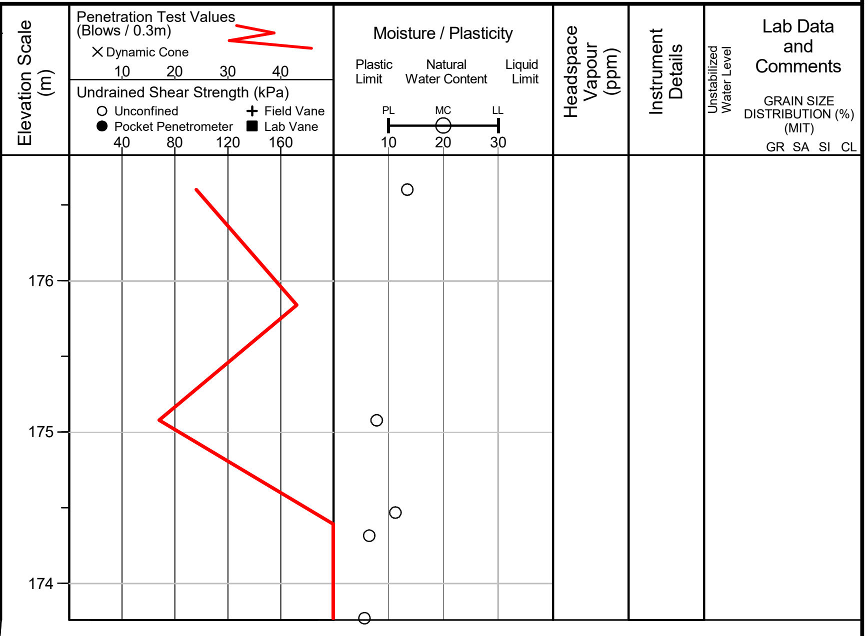
Position : E: 562213, N: 4928162 (UTM 17T)
Elevation Datum : Geodetic

Rig type : Track-mounted
Drilling Method : Solid stem augers

Elev Depth (m)	SOIL PROFILE Description	Graphic Log	SAMPLES Number	Type	SPT 'N' Value
176.8	**GROUND SURFACE**				
0	**FILL**, silty sand, trace to some gravel, trace clay, trace rootlets, compact to dense, brown, wet		1	SS	24
	...at 0.8 m, cobbles		2	SS	43
175.3 1.5	**SANDY SILT**, trace to some gravel, trace clay, compact to very dense, grey, wet (GLACIAL TILL)		3	SS	17
			4A 4B	SS	50 / 150mm
173.7 3.1	...at 3.0 m, cobbles		5	SS	50 / 25mm

END OF BOREHOLE
Auger refusal on possible bedrock

Water level was not measured due to unstabilized water depth.



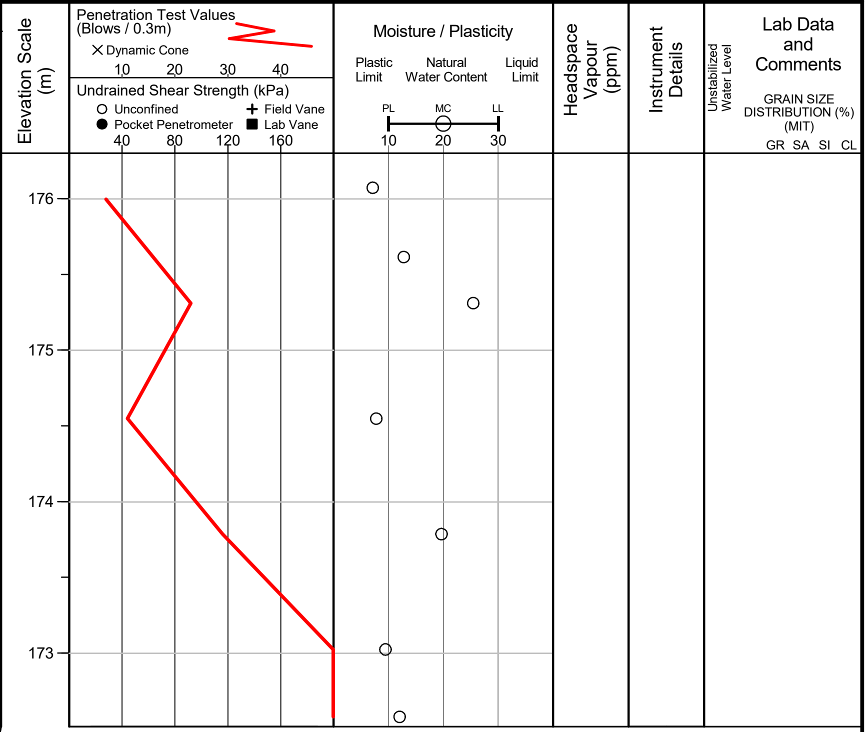
Terraprobe

LOG OF BOREHOLE 5

Project No. : 3-20-0117-01	Client : F.S. Collingwood Development Ltd.	Originated by : RS
Date started : November 5, 2020	Project : Collingwood Shipyards Block 6	Compiled by : HS
Sheet No. : 1 of 1	Location : Collingwood, ON	Checked by : OB

Position : E: 562226, N: 4928197 (UTM 17T)	Elevation Datum : Geodetic
Rig type : Track-mounted	Drilling Method : Solid stem augers

Depth Scale (m)	Elev Depth (m)	SOIL PROFILE Description	Graphic Log	SAMPLES Number	Type	SPT 'N' Value
	176.3	**GROUND SURFACE**				
0	175.8 0.5	450mm **CRUSHER RUN LIMESTONE**, trace sand, trace silt, trace clay, trace gravel, white, moist	▨	1A	SS	7
	175.4 0.9	**FILL**, silty sand, some gravel, trace clay, trace organics, loose, brown, moist	▨	1B		
1	174.8 1.5	**FILL**, silty sand, trace gravel, trace clay, compact, brown, wet	▨	2	SS	23
2	174.0 2.3	**SANDY SILT**, trace clay, trace gravel, compact, greyish brown, wet (GLACIAL TILL)		3	SS	11
	173.6 2.7	**SANDY SILT**, trace clay, trace gravel, compact, grey, wet		4	SS	29
3	172.5 3.8	**SANDY SILT**, trace clay, trace gravel, compact, greyish brown, wet (GLACIAL TILL)		5 6	SS SS	75 50 / 125mm



END OF BOREHOLE
Auger refusal on possible bedrock

Water level was not measured due to unstabilized water depth.

LOG OF BOREHOLE 6

Project No. : 3-20-0117-01
Client : F.S. Collingwood Development Ltd.
Originated by : RS

Date started : November 5, 2020
Project : Collingwood Shipyards Block 6
Compiled by : HS

Sheet No. : 1 of 1
Location : Collingwood, ON
Checked by : OB

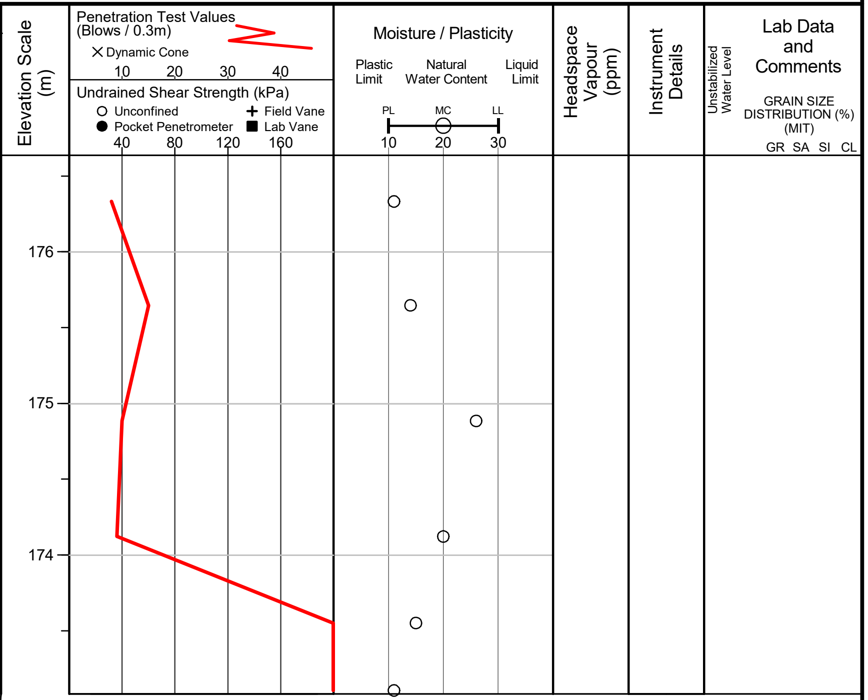
Position : E: 562211, N: 4928191 (UTM 17T)
Elevation Datum : Geodetic

Rig type : Track-mounted
Drilling Method : Solid stem augers

Elev Depth (m)	SOIL PROFILE Description	SAMPLES Number	Type	SPT 'N' Value
176.6	**GROUND SURFACE**			
	750mm **CRUSHER RUN LIMESTONE**, loose, light brown, moist	1	SS	8
175.8 0.8	**FILL**, silty sand, some gravel, trace clay, trace organics, concrete fragments, dark grey, wet	2	SS	15
	...at 1.5 m, black organics / peat inclusions	3	SS	10
		4	SS	9
173.6 3.0	**SANDY SILT**, with gravelly sand layers, very dense, grey, wet (GLACIAL TILL)	5	AS	50 / 75mm
173.0 3.6		6	SS	50 / 50mm

END OF BOREHOLE
Auger refusal on possible bedrock

Water level was not measured due to unstabilized water depth.



Terraprobe

LOG OF BOREHOLE 7

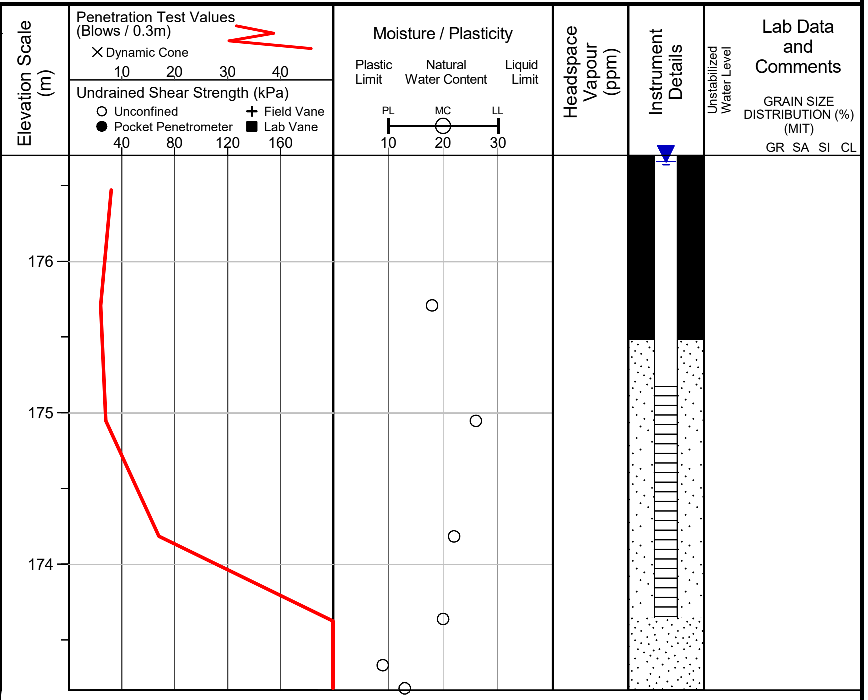
Project No. : 3-20-0117-01
Date started : November 5, 2020
Sheet No. : 1 of 1

Client : F.S. Collingwood Development Ltd.
Project : Collingwood Shipyards Block 6
Location : Collingwood, ON

Originated by : RS
Compiled by : HS
Checked by : OB

Position : E: 562215, N: 4928244 (UTM 17T)
Rig type : Track-mounted
Elevation Datum : Geodetic
Drilling Method : Solid stem augers

Depth Scale (m)	Elev Depth (m)	SOIL PROFILE Description	Graphic Log	SAMPLES Number	Type	SPT 'N' Value
	176.7	**GROUND SURFACE**				
0		**FILL**, silty sand, some gravel, trace clay, trace organics, loose, brown, wet		1	SS	8
1				2	SS	6
2				3	SS	7
	174.4 2.3	**SILTY SAND**, trace gravel, trace clay, compact to very dense, grey, wet (POSSBILE FILL)		4	SS	17
3				5A	SS	50 / 50mm
	173.3					
	173.2 3.5	**SANDY SILT**, some gravel, trace clay, very dense, grey, wet (GLACIAL TILL)		5B		
				6	AS	50 / 25mm



END OF BOREHOLE
Auger refusal on possible bedrock

Water level in the well is 0.4m above the ground surface on November 16, 2020.

Water level was not measured due to unstabilized water depth.

25 mm dia. piezometer installed.

WATER LEVEL READINGS

Date	Water Depth (m)	Elevation (m)
Apr 19, 2021	0.0	176.7

file: 3-20-0117 bh logs_corr bh elev.gpj

Terraprobe

LOG OF BOREHOLE 8

Project No. : 3-20-0117-01
Client : F.S. Collingwood Development Ltd.
Originated by : RS
Date started : November 5, 2020
Project : Collingwood Shipyards Block 6
Compiled by : HS
Sheet No. : 1 of 1
Location : Collingwood, ON
Checked by : OB

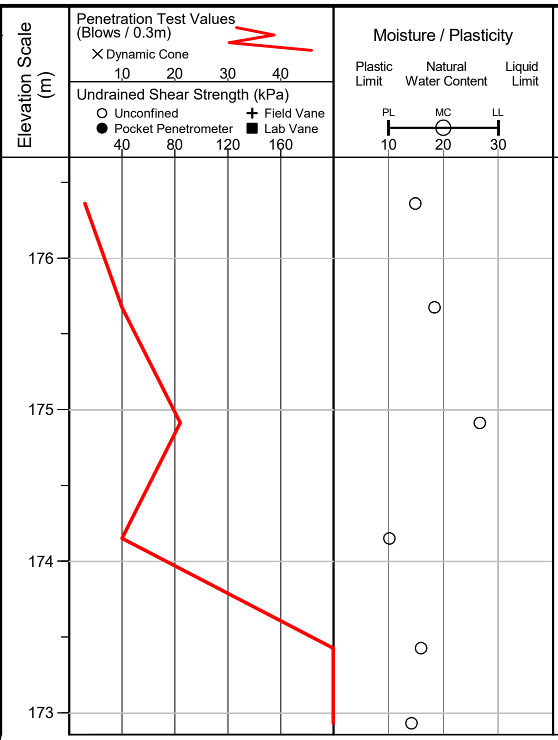
Position : E: 562203, N: 4928242 (UTM 17T)
Elevation Datum : Geodetic
Rig type : Track-mounted
Drilling Method : Solid stem augers

SOIL PROFILE

Elev Depth (m)	Description
176.7	**GROUND SURFACE**
175.9 0.8	750mm **CRUSHER RUN LIMESTONE**, some silt, some sand, trace clay, trace gravel, very dense, brown, wet
174.4 2.3	**FILL**, silty sand, some gravel, trace clay, trace organics, wood pieces, compact, brown, wet
172.9 3.8	**SILTY SAND to SANDY SILT**, trace to some gravel, trace clay, loose to very dense, grey, wet (GLACIAL TILL)

SAMPLES

Number	Type	SPT 'N' Value	Lab Data and Comments GRAIN SIZE DISTRIBUTION (%) (MIT) GR SA SI CL
1	SS	3	
2	SS	10	
3	SS	21	
4	SS	10	17 40 35 8
5	SS	78 / 225mm	
6	AS	50 / 150mm	



Headspace Vapour (ppm) | Instrument Details | Unstabilized Water Level

END OF BOREHOLE
Auger refusal on possible bedrock

Water level was not measured due to unstabilized water depth.

Terraprobe

LOG OF BOREHOLE 102

Project No. : 3-20-0117-01
Date started : March 3, 2021
Sheet No. : 1 of 1

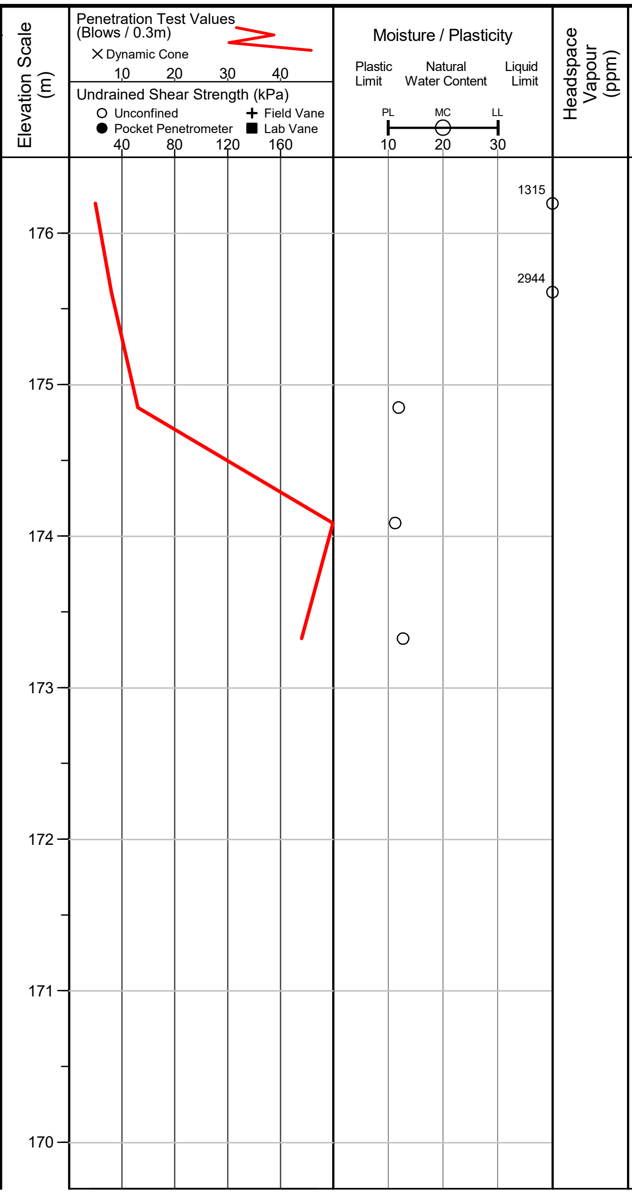
Client : F.S. Collingwood Development Ltd.
Project : Collingwood Shipyards Block 6
Location : Collingwood, ON

Originated by : F/R
Compiled by : CM
Checked by : OB

Position : E: 562209, N: 4928274 (UTM 17T)
Rig type : Track-mounted

Elevation Datum : Geodetic
Drilling Method : Solid stem augers

SOIL PROFILE		SAMPLES						Lab Data and Comments			
Elev Depth (m)	Description	Graphic Log	Number	Type	SPT 'N' Value	Instrument Details	Unstabilized Water Level	GR	SA	SI	CL
176.5	**GROUND SURFACE**										
	FILL, silty sand, trace clay, trace gravel, trace wood fragments, trace rootlets, loose to compact, grey, wet	▨	1	SS	5						
			2	SS	8						
			3	SS	13						
174.3 2.2	**SANDY SILT to SILTY SAND**, trace clay, trace gravel, dense to very dense, grey, wet (GLACIAL TILL)		4	SS	79						
			5	SS	44			9	40	49	2
172.8 3.7	**LIMESTONE** (See rock core log for details)		1	RUN							
			2	RUN							
			3	RUN							
169.7 6.8											



END OF BOREHOLE

Borehole contained drill water upon completion of drilling. Unstabilized water level and cave not measured.

Terraprobe

LOG OF BOREHOLE 104

Project No. : 3-20-0117-01
Date started : March 3, 2021
Sheet No. : 1 of 1

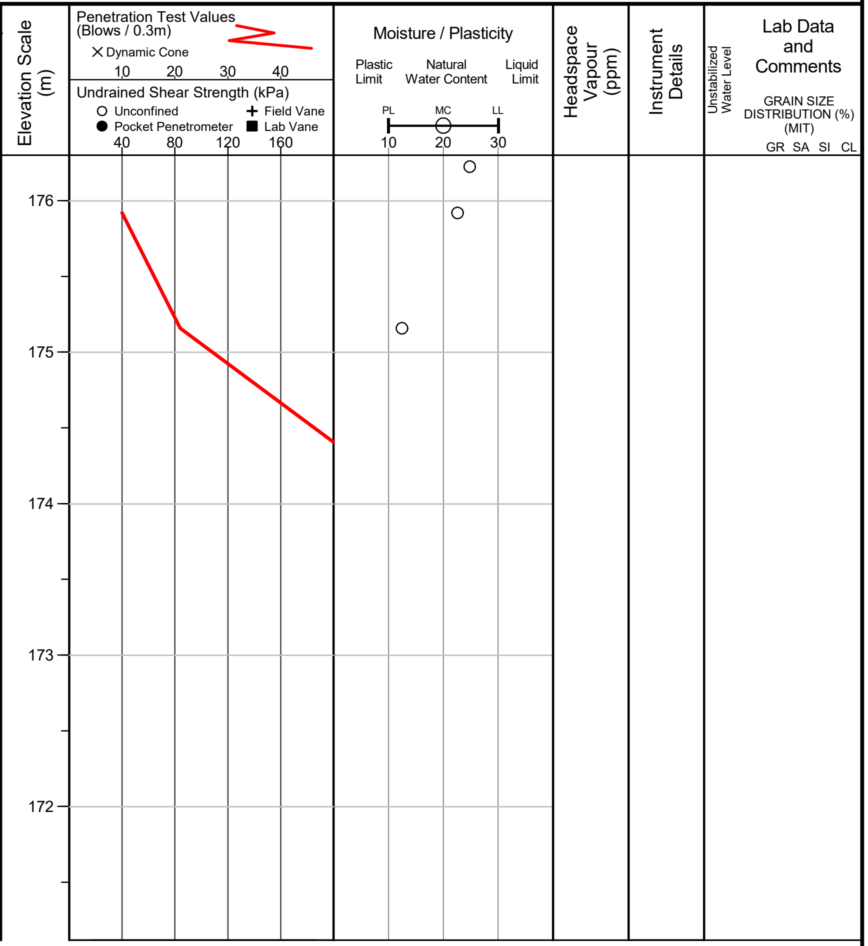
Client : F.S. Collingwood Development Ltd.
Project : Collingwood Shipyards Block 6
Location : Collingwood, ON

Originated by : F/R
Compiled by : CM
Checked by : OB

Position : E: 562198, N: 4928214 (UTM 17T)
Rig type : Track-mounted

Elevation Datum : Geodetic
Drilling Method : Solid stem augers

Depth Scale (m)	Elev Depth (m)	Description	Number	Type	SPT 'N' Value
	176.3	**GROUND SURFACE**			
0		**FILL**, silty sand, trace clay, trace gravel, trace organics, concrete fragments, loose to compact, dark grey, wet	1	AS	
			2	SS	10
1			3	SS	21
	174.6 1.7	**SANDY SILT to SILTY SAND**, trace clay, trace gravel, very dense, grey, wet (GLACIAL TILL)	4	SS	68 / 275mm
2	174.2 2.1	**LIMESTONE** (See rock core log for details)			
3			1	RUN	
4			2	RUN	
5	171.1 5.2				



END OF BOREHOLE

Borehole contained drill water upon completion of drilling. Unstabilized water level and cave not measured.

LOG OF BOREHOLE 108

Project No. : 3-20-0117-01	Client : F.S. Collingwood Development Ltd.	Originated by : F/R
Date started : March 3, 2021	Project : Collingwood Shipyards Block 6	Compiled by : CM
Sheet No. : 1 of 1	Location : Collingwood, ON	Checked by : OB

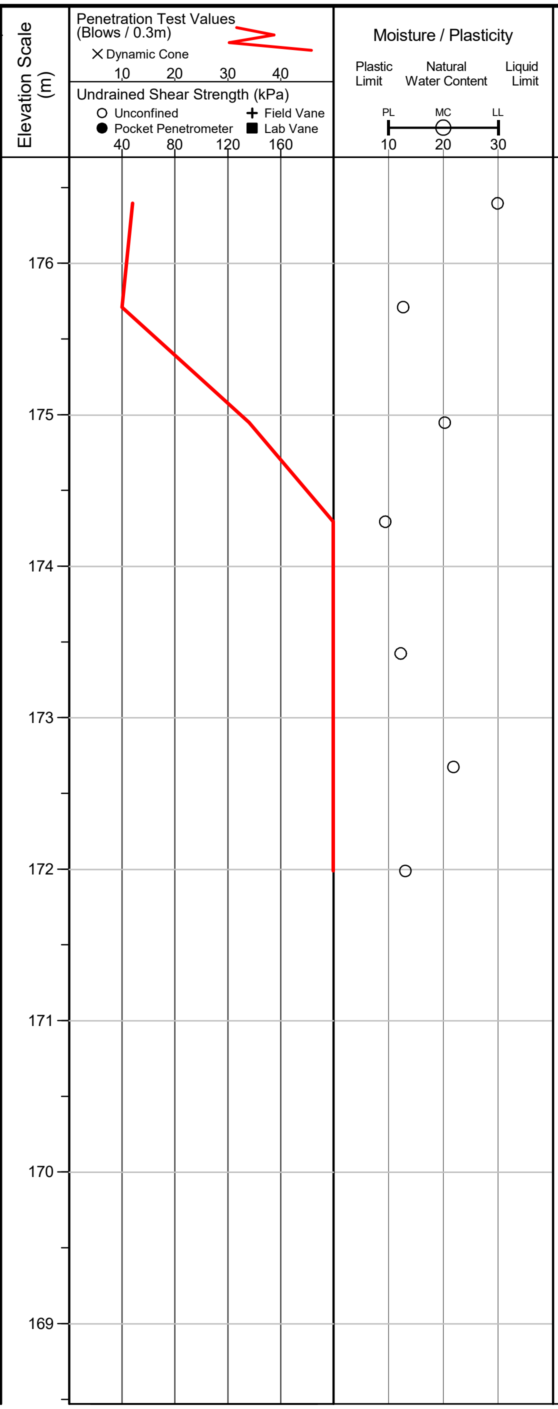
Position : E: 562283, N: 4928169 (UTM 17T)

Elevation Datum : Geodetic

Rig type : Track-mounted

Drilling Method : Solid stem augers

Elev Depth (m)	Description	Sample Number	Type	SPT 'N' Value	Lab Data and Comments GRAIN SIZE DISTRIBUTION (%) (MIT) GR SA SI CL
176.7	**GROUND SURFACE**				
	FILL, silty sand, trace clay, trace gravel, trace organics, trace concrete, compact, grey, wet	1	SS	12	
		2	SS	10	
175.2 1.5	**SAND**, some silt, dense, grey, wet	3	SS	34	0 87 13 0
174.4 2.3	**SANDY SILT to SILTY SAND**, trace cobbles, trace gravel, dilatant, very dense, grey, wet (GLACIAL TILL)	4	SS	50 / 90mm	
		5	SS	83	
		6	SS	86 / 275mm	
		7	SS	50 / 75mm	
171.5 5.2	**LIMESTONE** (See rock core log for details)	1	RUN		
		2	RUN		
168.5 8.2					



Headspace Vapour (ppm)

Instrument Details

Unstabilized Water Level

END OF BOREHOLE

Borehole contained drill water upon completion of drilling. Unstabilized water level and cave not measured.

file: 3-20-0117 bh logs_corr bh elev.gpj

ROCK CORE LOG 102

Project No. : 3-20-0117-01
Client : F.S. Collingwood Development Ltd.
Originated by : F/R

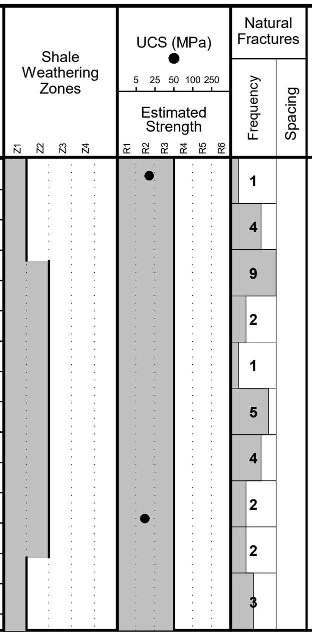
Date started : March 3, 2021
Project : Collingwood Shipyards Block 6
Compiled by : CM

Sheet No. : 1 of 1
Location : Collingwood, ON
Checked by : OB

Position : E: 562209, N: 4928274 (UTM 17T)
Elevation Datum : Geodetic

Rig type : Track-mounted
Drilling Method : Solid stem augers

Depth (m)	GENERAL DESCRIPTION	Run	Elev Depth (m)	Recovery	Elevation (m)	Laboratory Testing	Comments
	Rock coring started at 3.7m below grade		**172.8**				
4	**LIMESTONE**, grey, medium bedded to thinly bedded, medium strong; closed to gapped	R1	3.7 3.8	TCR = **100%** SCR = **100%** RQD = **0%**		**3.8m to 4.0m:** UCS = 18.81 MPa	**4.1m:** *approx. 310mm long fractured zone*
5		R2		TCR = **100%** SCR = **77%** RQD = **58%**	172		
			171.2 5.3		171		
6		R3		TCR = **97%** SCR = **88%** RQD = **62%**	170	**6.1m to 6.2m:** UCS = 14.23 MPa	
			169.7 **6.8m**				



END OF COREHOLE

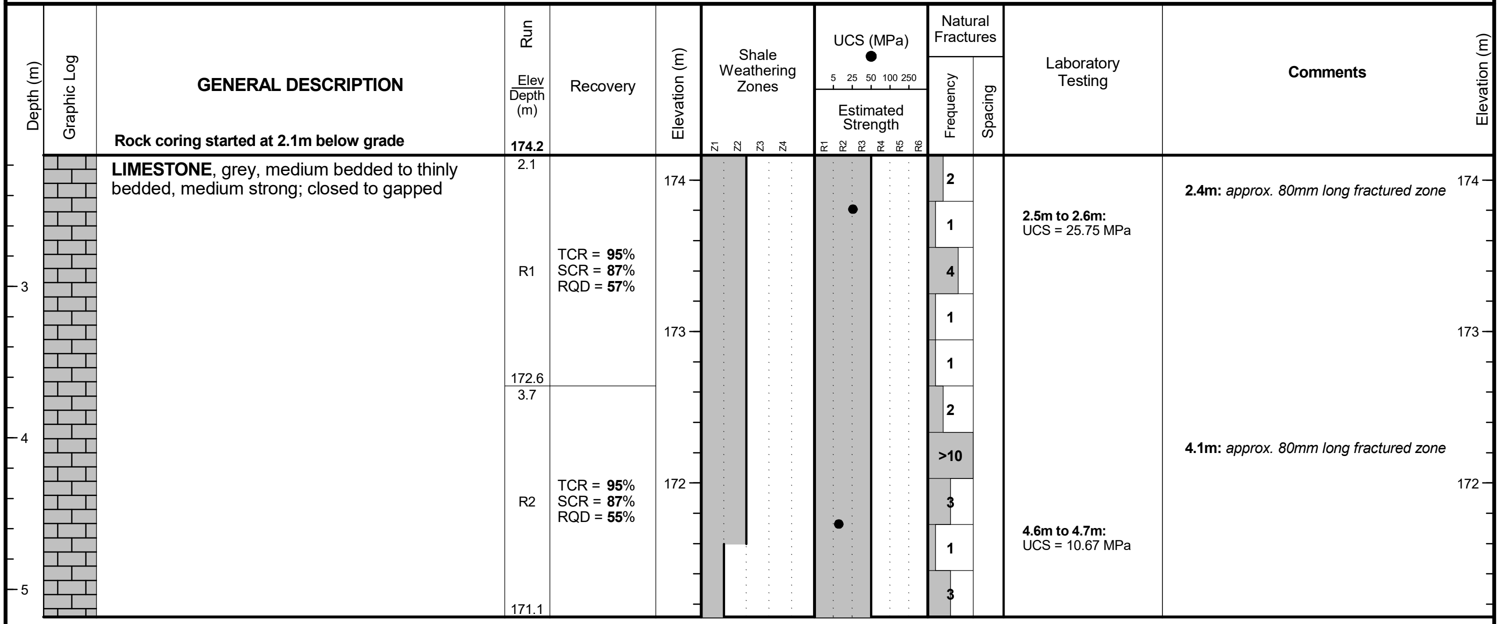
file: 3-20-0117 bh logs_corr bh elev.gpj

Terraprobe

ROCK CORE LOG 104

Project No. : 3-20-0117-01
Client : F.S. Collingwood Development Ltd.
Originated by : F/R
Date started : March 3, 2021
Project : Collingwood Shipyards Block 6
Compiled by : CM
Sheet No. : 1 of 1
Location : Collingwood, ON
Checked by : OB

Position : E: 562198, N: 4928214 (UTM 17T)
Elevation Datum : Geodetic
Rig type : Track-mounted
Drilling Method : Solid stem augers



Depth (m)	GENERAL DESCRIPTION	Run	Elev Depth (m)	Recovery	Natural Fractures Frequency	Laboratory Testing	Comments
	Rock coring started at 2.1m below grade		174.2				
2.1–3.7	**LIMESTONE**, grey, medium bedded to thinly bedded, medium strong; closed to gapped	R1	2.1 172.6	TCR = **95%** SCR = **87%** RQD = **57%**	2 1 4 1 1	**2.5m to 2.6m:** UCS = 25.75 MPa	**2.4m:** *approx. 80mm long fractured zone*
3.7–5.2		R2	3.7 171.1	TCR = **95%** SCR = **87%** RQD = **55%**	2 >10 3 1 3	**4.6m to 4.7m:** UCS = 10.67 MPa	**4.1m:** *approx. 80mm long fractured zone*

5.2m

END OF COREHOLE

file: 3-20-0117 bh logs_corr bh elev.gpj

ROCK CORE LOG 108

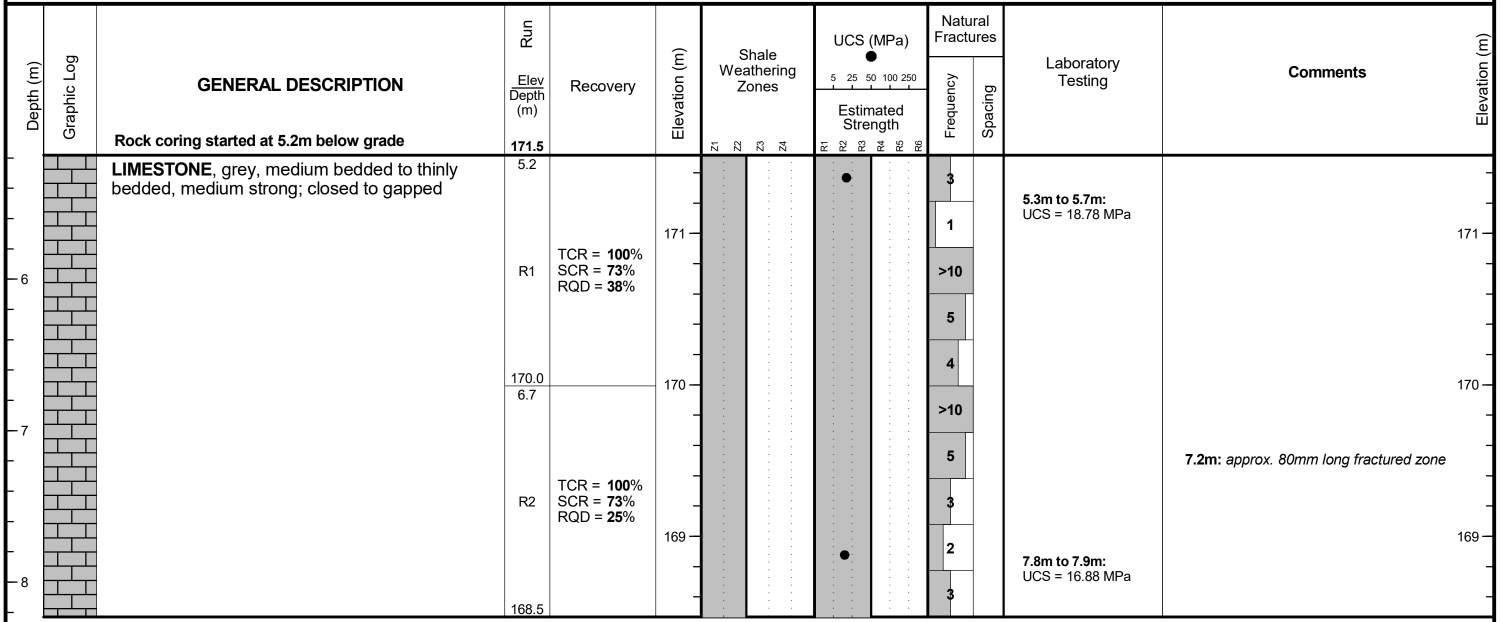
Project No. : 3-20-0117-01
Date started : March 3, 2021
Sheet No. : 1 of 1

Client : F.S. Collingwood Development Ltd.
Project : Collingwood Shipyards Block 6
Location : Collingwood, ON

Originated by : F/R
Compiled by : CM
Checked by : OB

Position : E: 562283, N: 4928169 (UTM 17T)
Rig type : Track-mounted

Elevation Datum : Geodetic
Drilling Method : Solid stem augers



GENERAL DESCRIPTION

Rock coring started at 5.2m below grade

LIMESTONE, grey, medium bedded to thinly bedded, medium strong; closed to gapped

Run	Elev / Depth (m)	Recovery
	171.5	
R1	5.2	TCR = **100%** SCR = **73%** RQD = **38%**
	170.0 6.7	
R2		TCR = **100%** SCR = **73%** RQD = **25%**
	168.5 **8.2m**	

Natural Fractures – Frequency: 3, 1, >10, 5, 4, >10, 5, 3, 2, 3

Laboratory Testing

5.3m to 5.7m:
UCS = 18.78 MPa

7.8m to 7.9m:
UCS = 16.88 MPa

Comments

7.2m: *approx. 80mm long fractured zone*

END OF COREHOLE

APPENDIX B

TERRAPROBE INC.

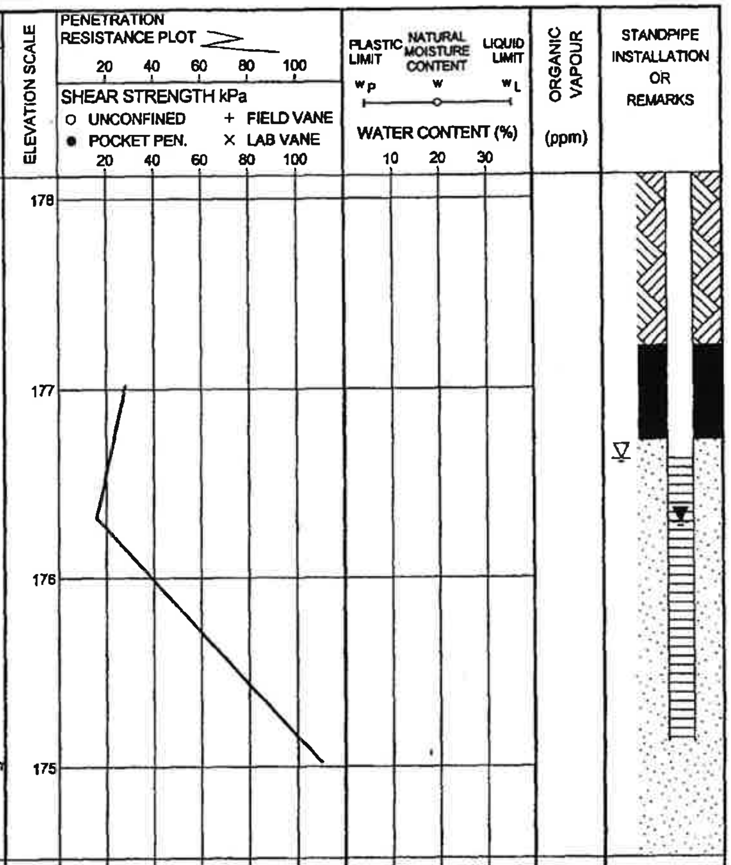
Terraprobe

LOG OF BOREHOLE MW 03-10

PROJECT: Collingwood Ship Yard - CSL Lands
LOCATION: Collingwood, Ontario
CLIENT: CSL Equity Investments

DATE: 09 June 2003
EQUIPMENT: Bombardier
ELEVATION DATUM: Geodetic
FILE: 1-01-0400

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES
178.1	Ground Surface			
0.0	Loose to Compact Grey/Brown wet **FILL** - Sand and Silt, some gravel, trace cobbles	1	AS	
		2	SS	28
176.6 1.5	Compact to Very Dense Grey wet **FINE SAND TO SILTY SAND** trace silt, some gravel	3	SS	16
		4	SS	50/8cm
174.5 3.6	End of Borehole Auger Refusal @ 3.6m			



NOTES:
Borehole was open and water level at 1.5m upon completion of drilling. Water level in standpipe at 1.84mbgl on June 17, 2003.

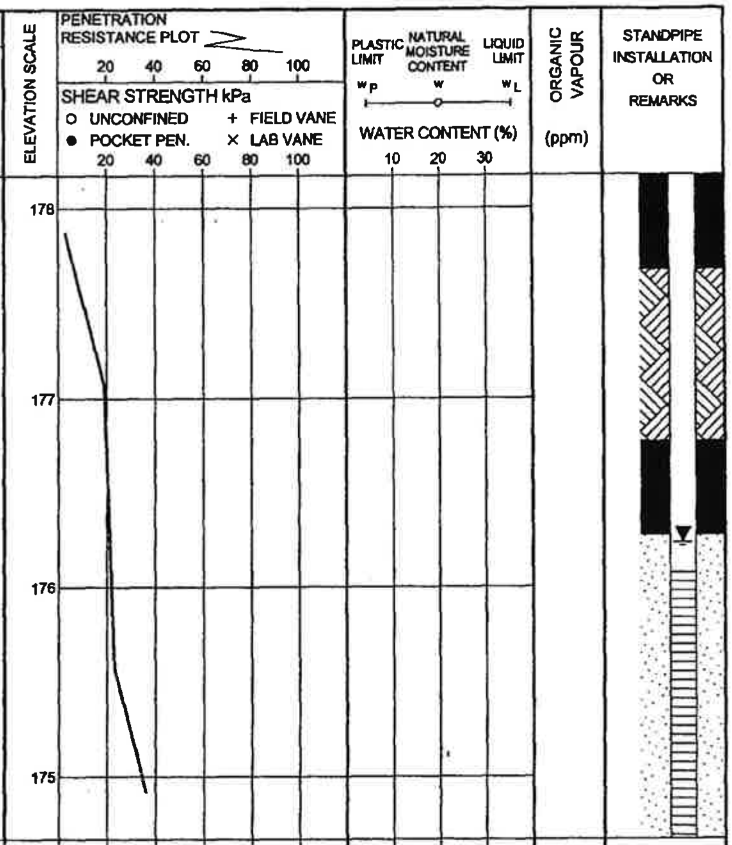
Terraprobe

LOG OF BOREHOLE MW 03-14

PROJECT: Collingwood Ship Yard - CSL Lands
LOCATION: Collingwood, Ontario
CLIENT: CSL Equity Investments

DATE: 21 July 2003
EQUIPMENT: Bombardier
ELEVATION DATUM: Geodetic
FILE: 1-01-0400

SOIL PROFILE			SAMPLES		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES
178.2	Ground Surface				
0.0	Very Loose to Compact Brown/Black moist **FILL** - Sand and Silt, some cinders, gravel	▨	1	SS	3
			2	SS	19
176.7 1.5	Compact to Dense Grey wet **SILTY SAND TO SAND** trace gravel		3	AS	21
			4	SS	23
			5	SS	36
174.7 3.5	End of Borehole				



NOTES:
Borehole was open upon completion of drilling. Water level in standpipe at 1.94mbgl on August 1, 2003.

Terraprobe

LOG OF BOREHOLE MW 03-15

PROJECT: Collingwood Ship Yard - CSL Lands
LOCATION: Collingwood, Ontario
CLIENT: CSL Equity Investments

DATE: 21 July 2003
EQUIPMENT: Bombardier
ELEVATION DATUM: Geodetic
FILE: 1-01-0400

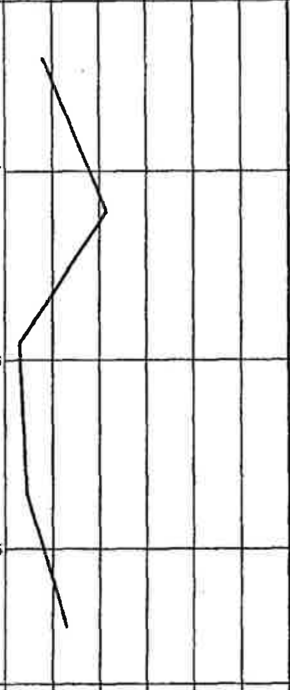
SOIL PROFILE			SAMPLES			ELEVATION SCALE
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	
177.9	Ground Surface					
0.0	Loose to Dense Brown/Black moist FILL - Sand, Gravel, some ash, wood	▩	1	SS	16	
			2	SS	43	177
			3	SS	6	176
175.6 / 2.3	Loose to Compact Grey wet SAND trace gravel	░	4	SS	9	175
			5	SS	26	
174.3 / 3.6	End of Borehole					

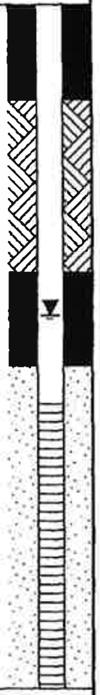
PENETRATION RESISTANCE PLOT 20 40 60 80 100
SHEAR STRENGTH kPa
○ UNCONFINED + FIELD VANE
● POCKET PEN. × LAB VANE
20 40 60 80 100

PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L
WATER CONTENT (%) 10 20 30

ORGANIC VAPOUR (ppm)

STANDPIPE INSTALLATION OR REMARKS





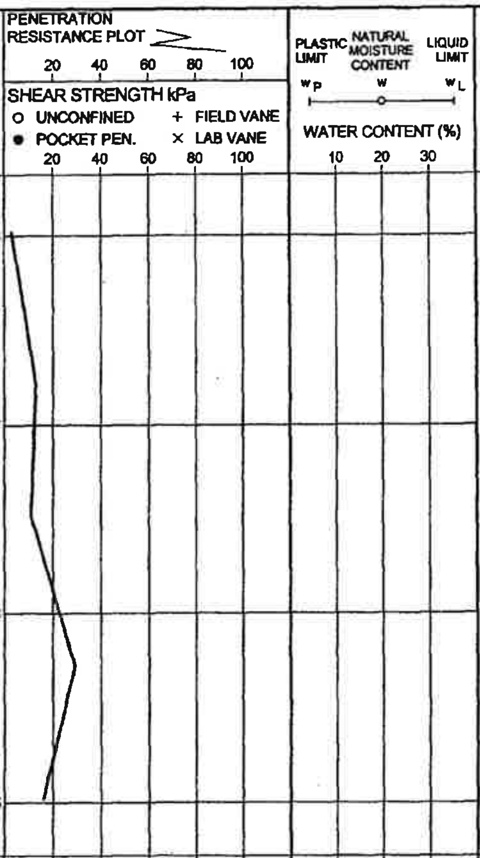
NOTES:
Borehole was open upon completion of drilling. Water level in standpipe at 1.63mbgl on August 1, 2003.

Terraprobe

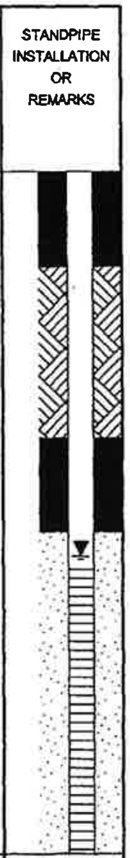
LOG OF BOREHOLE MW 03-16

PROJECT: Collingwood Ship Yard - CSL Lands
LOCATION: Collingwood, Ontario
CLIENT: CSL Equity Investments
DATE: 21 July 2003
EQUIPMENT: Bombardier
ELEVATION DATUM: Geodetic
FILE: 1-01-0400

SOIL PROFILE			SAMPLES			
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE
178.3	Ground Surface					
0.0	Loose to Compact Brown moist **FILL** - Sandy Silt to Sand, trace clay and gravel	▩	1	SS	3	178
			2	SS	13	177
176.8 1.5	Compact Grey moist **SAND** trace gravel, clay	░	3	SS	11	
			4	SS	29	176
			5	SS	16	175
174.7 3.6	End of Borehole					



ORGANIC VAPOUR (ppm)



NOTES:
Borehole was open upon completion of drilling. Water level in standpipe at 2.03mbgl on August 1, 2003.

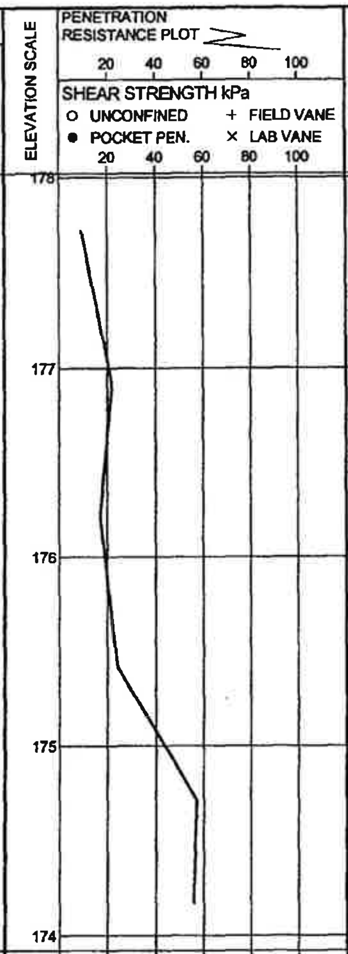
Sheet 1 of 1

Terraprobe

LOG OF BOREHOLE MW 03-17

PROJECT: Collingwood Ship Yard - CSL Lands
LOCATION: Collingwood, Ontario
CLIENT: CSL Equity Investments
DATE: 21 July 2003
EQUIPMENT: Bombardier
ELEVATION DATUM: Geodetic
FILE: 1-01-0400

SOIL PROFILE		SAMPLES		
ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES
178.0	Ground Surface			
0.0	Loose to Compact Brown/Grey moist **FILL** - Sand, trace gravel and silt, wood chips	1	SS	9
		2	SS	22
		3	SS	17
		4	SS	24
175.0 3.0	Compact to Very Dense Grey moist to wet **SAND** some gravel and silt	5	SS	57
		6	SS	56
173.9 4.1	End of Borehole			

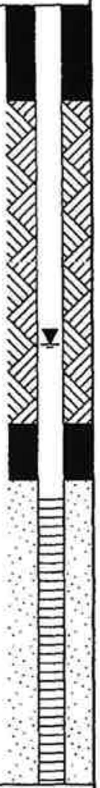


PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L

WATER CONTENT (%) 10 20 30

ORGANIC VAPOUR (ppm)

STANDPIPE INSTALLATION OR REMARKS

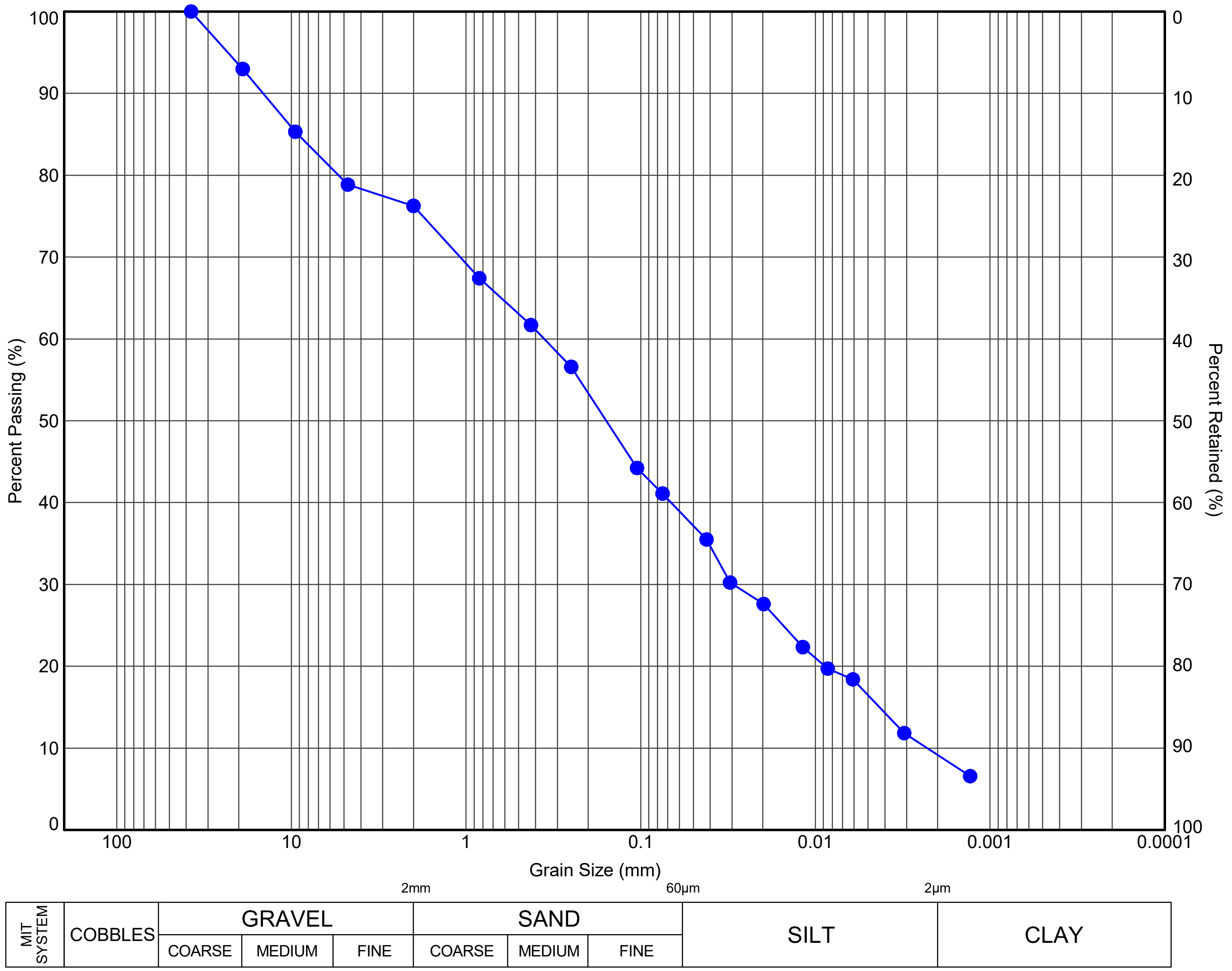


NOTES:
Borehole was open upon completion of drilling. Water level in standpipe at 1.78mbgl on August 1, 2003.

Sheet 1 of 1

APPENDIX C

TERRAPROBE INC.



					MIT SYSTEM				
	Hole ID	Sample	Depth (m)	Elev. (m)	Gravel (%)	Sand (%)	Silt (%)	Clay (%)	*(Fines, %)*
●	**1**	SS3	1.8	175.3	24	37	30	9	

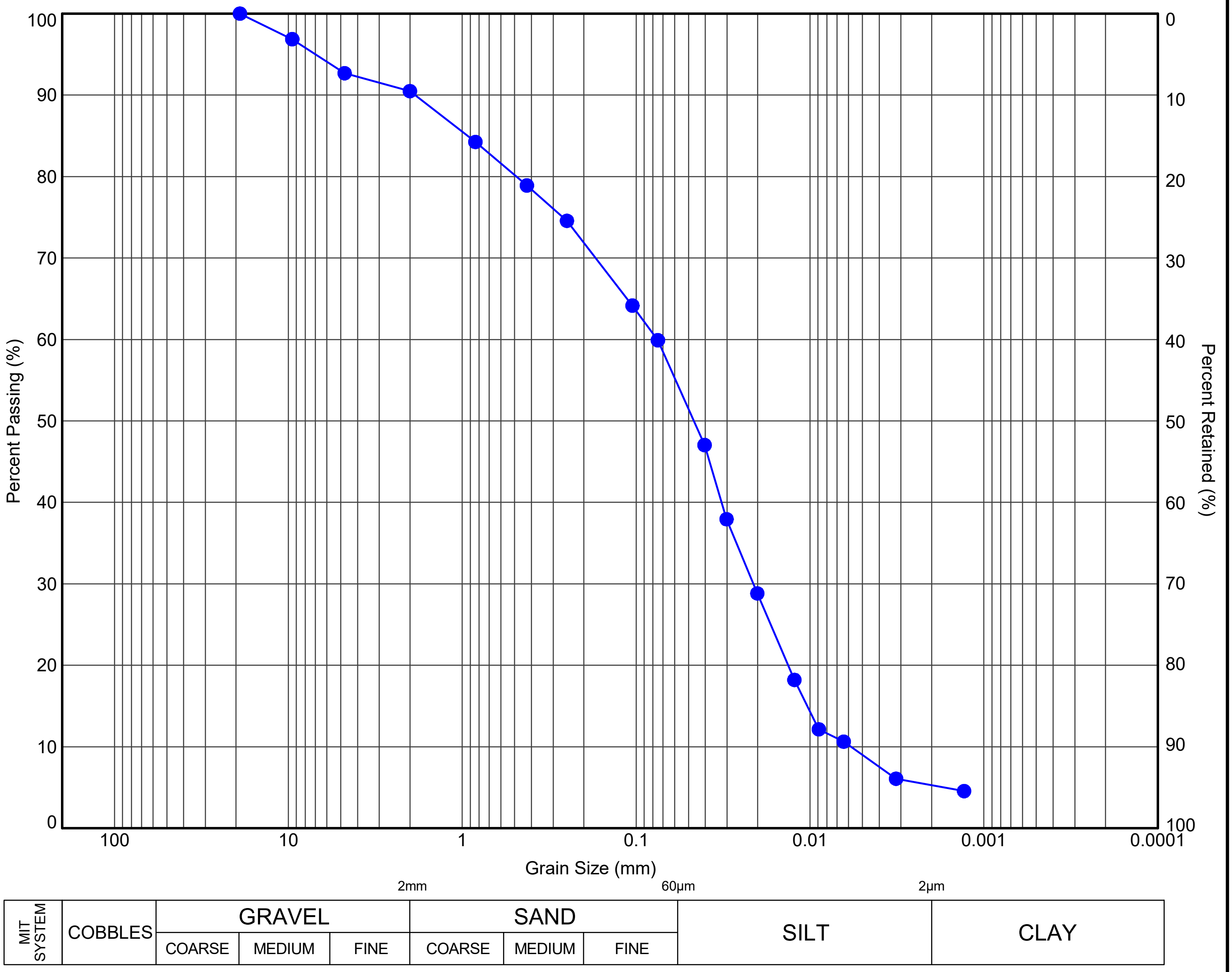
Terraprobe

11 Indell Lane, Brampton Ontario L6T 3Y3
(905) 796-2650

Title:

GRAIN SIZE DISTRIBUTION
GRAVELLY SILTY SAND, TRACE CLAY

File No.: **3-20-0117-01**



					MIT SYSTEM				
	Hole ID	Sample	Depth (m)	Elev. (m)	Gravel (%)	Sand (%)	Silt (%)	Clay (%)	*(Fines, %)*
●	**3**	4B	2.6	174.1	10	35	50	5	

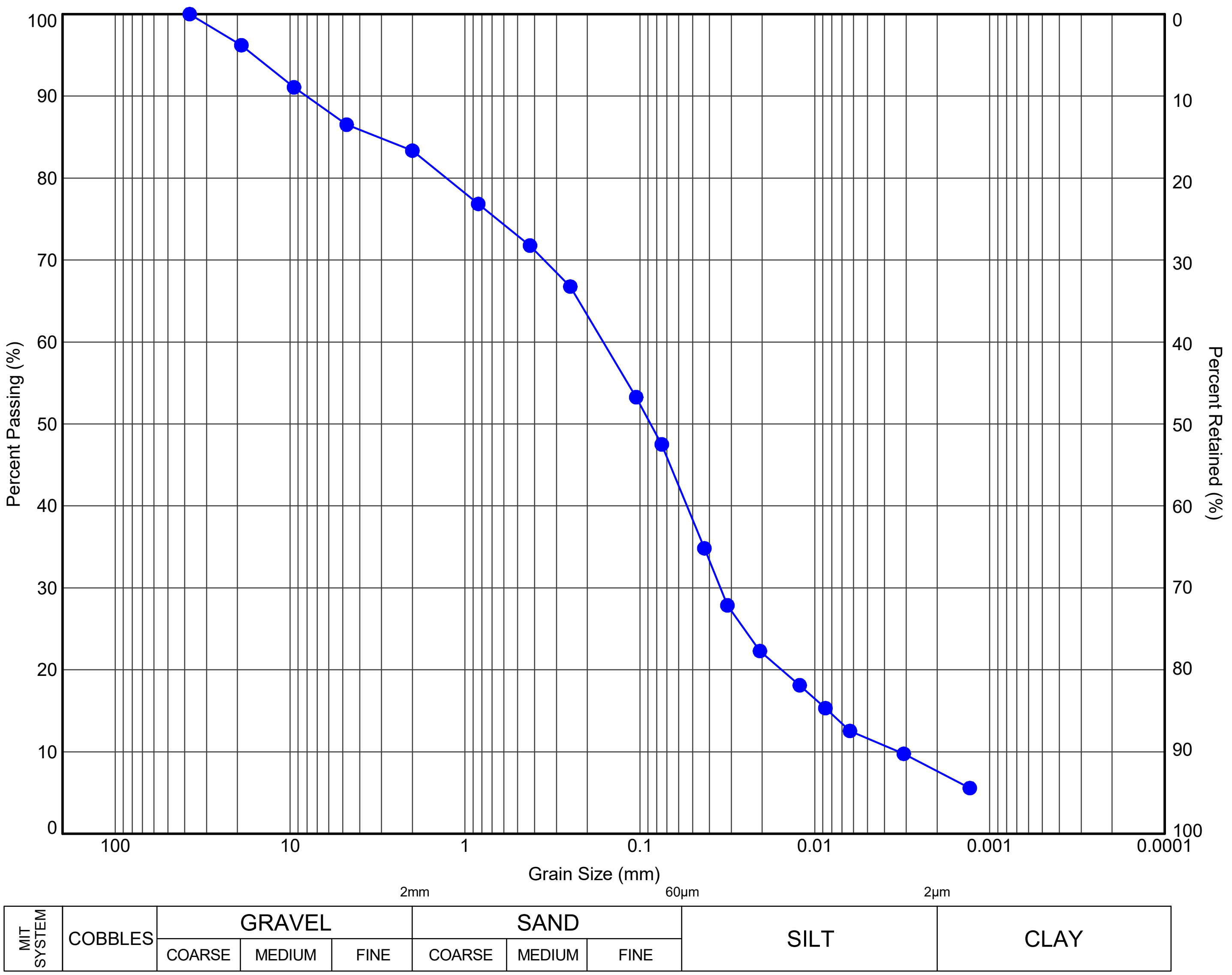
Terraprobe

11 Indell Lane, Brampton Ontario L6T 3Y3
(905) 796-2650

Title:

GRAIN SIZE DISTRIBUTION
SANDY SILT, GRAVELLY, TRACE CLAY

File No.: **3-20-0117-01**



					MIT SYSTEM				
	Hole ID	Sample	Depth (m)	Elev. (m)	Gravel (%)	Sand (%)	Silt (%)	Clay (%)	*(Fines, %)*
●	**8**	SS4	2.5	174.2	17	40	35	8	

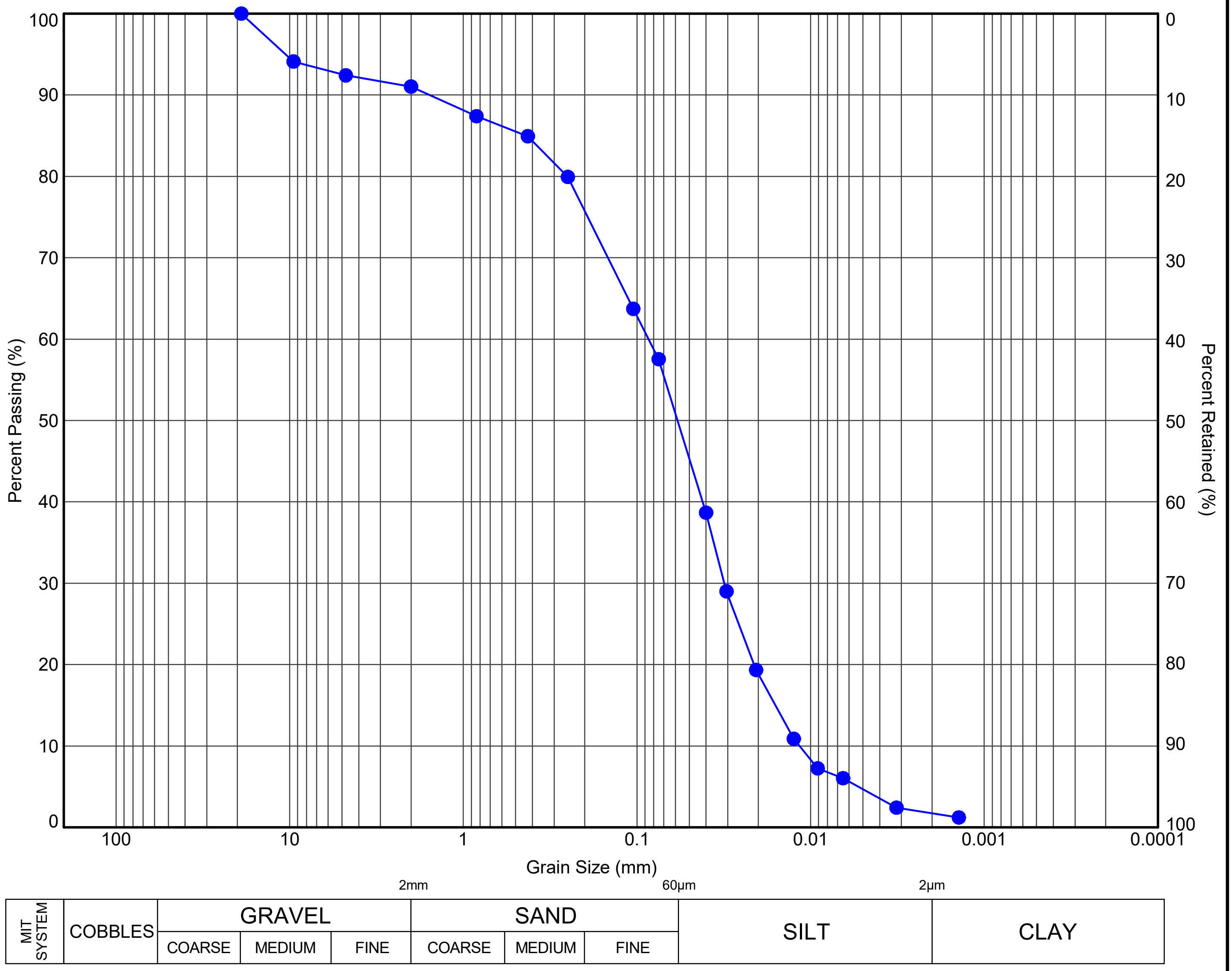
Terraprobe

11 Indell Lane, Brampton Ontario L6T 3Y3
(905) 796-2650

Title:

GRAIN SIZE DISTRIBUTION
SILTY SAND, SOME GRAVEL, TRACE CLAY

File No.: **3-20-0117-01**



MIT SYSTEM

	Hole ID	Sample	Depth (m)	Elev. (m)	Gravel (%)	Sand (%)	Silt (%)	Clay (%)	*(Fines, %)*
●	**102**	SS5	3.2	173.3	9	40	49	2	

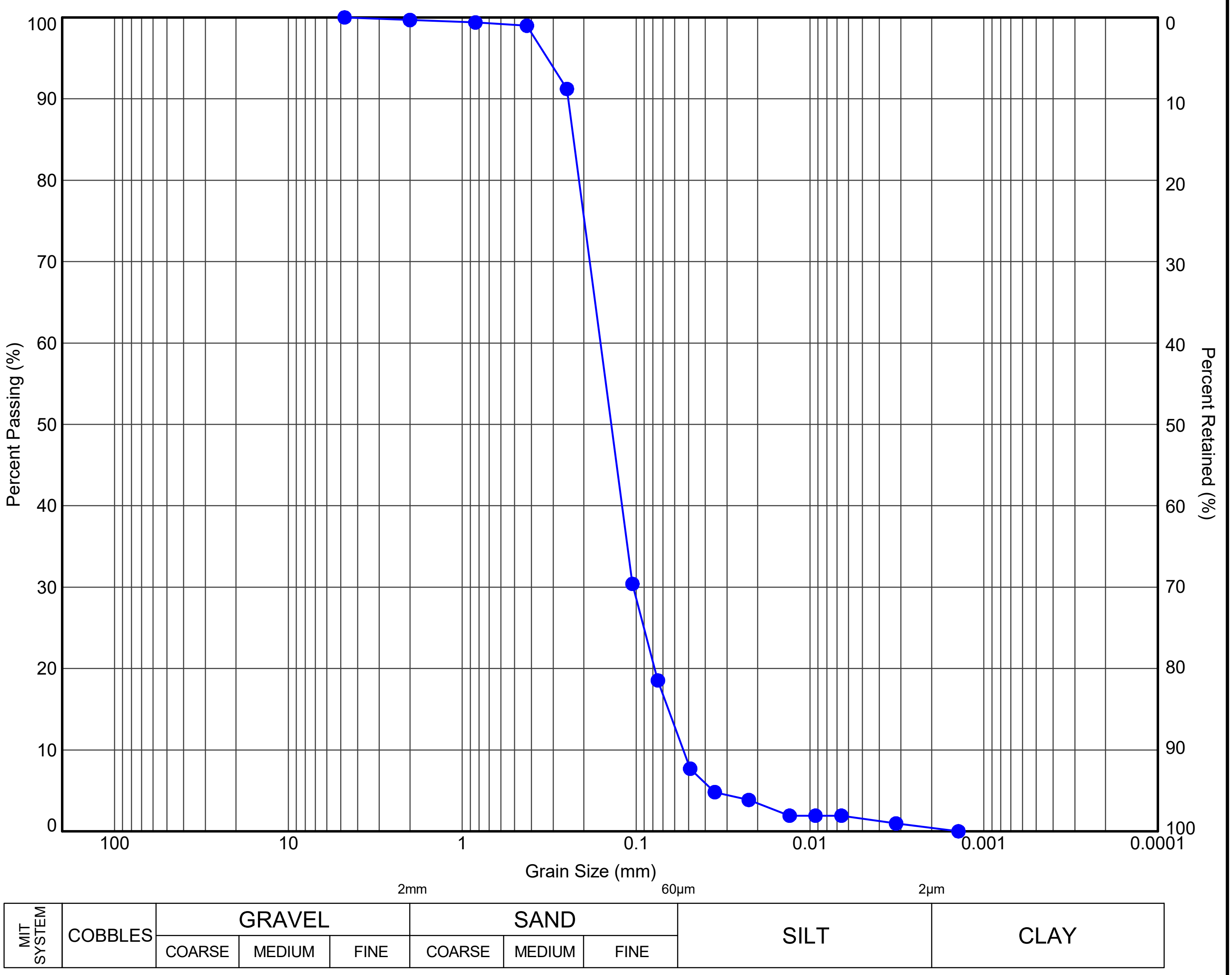
Terraprobe

11 Indell Lane, Brampton Ontario L6T 3Y3
(905) 796-2650

Title:

GRAIN SIZE DISTRIBUTION
SILT AND SAND, TRACE GRAVEL, TRACE CLAY

File No.: **3-20-0117-01**



					MIT SYSTEM				
	Hole ID	Sample	Depth (m)	Elev. (m)	Gravel (%)	Sand (%)	Silt (%)	Clay (%)	*(Fines, %)*
●	**108**	SS3	1.8	174.9	0	87	13	0	

Terraprobe

11 Indell Lane, Brampton Ontario L6T 3Y3
(905) 796-2650

Title:

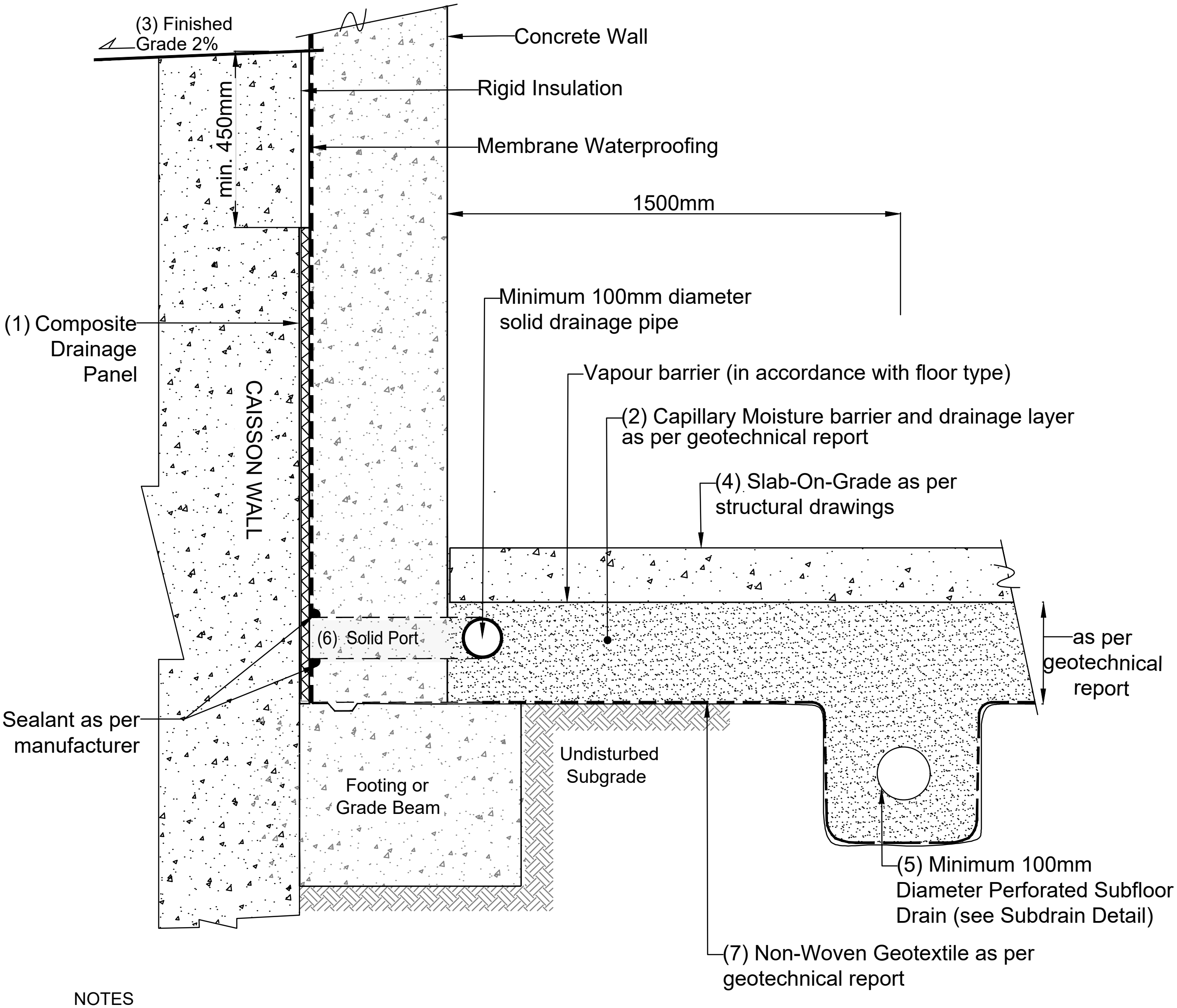
GRAIN SIZE DISTRIBUTION
SAND, SOME SILT

File No.:

3-20-0117-01

APPENDIX D

TERRAPROBE INC.



NOTES

1) Prefabricated composite drainage panels to consist of Miradrain 6000, or approved equivalent. Panels should provide continuous cover as per manufacturer's requirements.

2) Capillary moisture barrier/drainage layer to consist of a minimum 200mm layer of 19mm clear stone (OPSS. MUNI 1004), or as indicated in geotechnical report, compacted to a dense state. Upper 50mm can be replaced with Granular “A” (OPSS. MUNI 1010) compacted to 98% SPMDD where vehicular traffic is required. A vapour barrier may be required depending on floor type.

3) Exterior finished grade away from wall at a minimum grade of 2% for min. 1.2m.

4) Building floor slab-on-grade shall not be structurally connected to foundation wall or footing.

5) Subfloor drain invert to be a minimum of 300mm below underside of floor slab, to be set in parallel rows, one way, and at the spacing specified in the geotechnical report. Don't connect subfloor drains to perimeter drains.

6) Embedded ports to be set a distance of maximum 3m on-centre. Each port to have a minimum cross-sectional area of 1500mm². Perimeter drainage must be collected and conveyed directly to the building sumps in solidpipe.

7) When the subgrade consists of a cohesionless soil, the subgrade must be separated from the subfloor drainage layer using a non-woven geotextile (Terrafix 360R or approved equivalent).

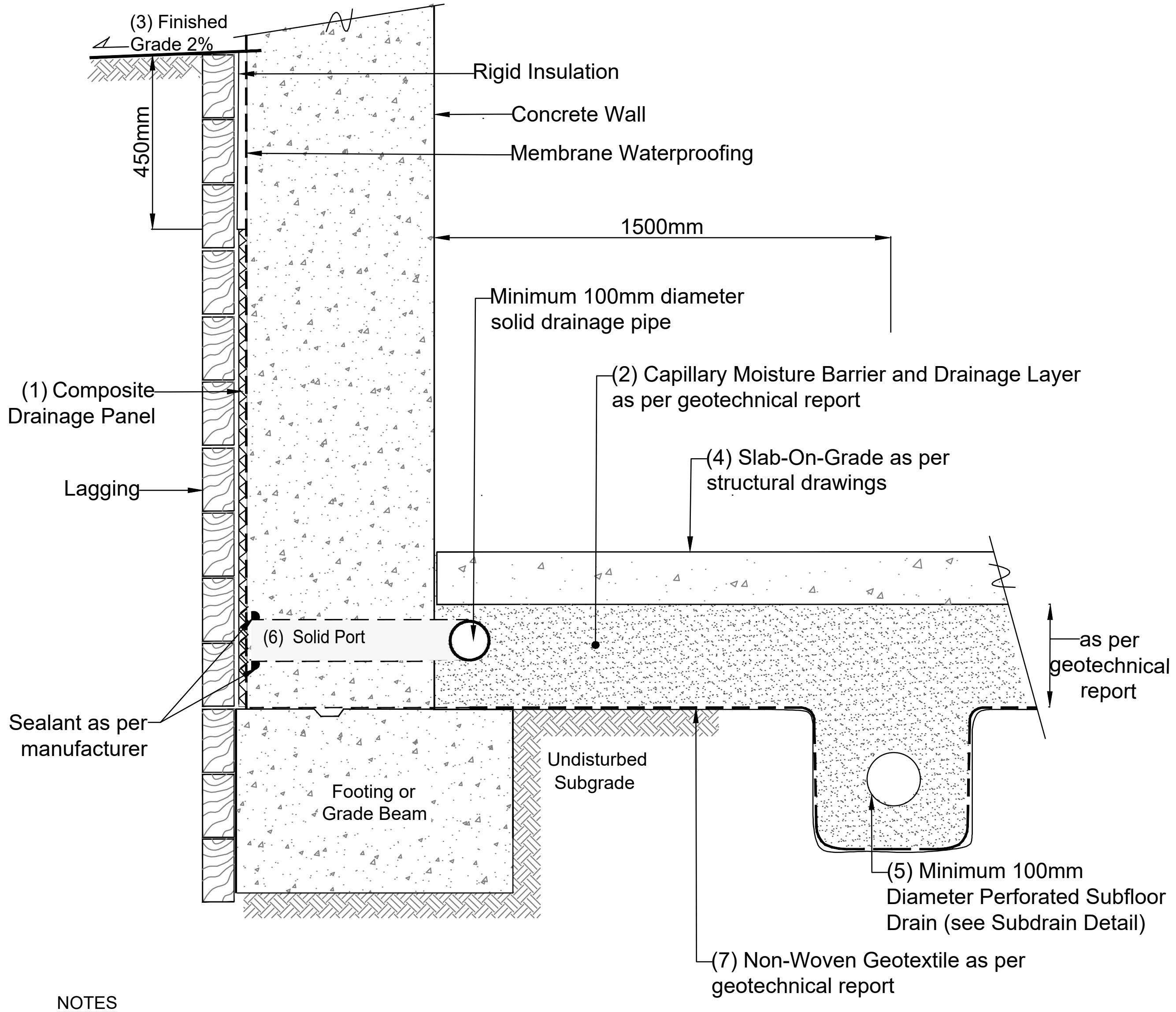
8) Geotechnical report contains specific details. Final detail must be reviewed before system is considered acceptable to use.

N.T.S

Terraprobe
11 Indell Lane, Brampton, Ontario, L6T 3Y3
Tel: (905) 796-2650 Fax: (905) 796-2250

Title:

SCHEMATIC BASEMENT DRAINAGE DETAIL
CAISSON WALL SHORING SYSTEM
(ONE-SIDED WALL CONSTRUCTION)



NOTES

1) Prefabricated composite drainage panels to consist of Miradrain 6000, or approved equivalent. Panels should provide continuous cover as per manufacturer's requirements.

2) Capillary moisture barrier/drainage layer to consist of a minimum 200mm layer of 19mm clear stone (OPSS. MUNI 1004), or as indicated in geotechnical report, compacted to a dense state. Upper 50mm can be replaced with Granular “A” (OPSS. MUNI 1010) compacted to 98% SPMDD where vehicular traffic is required. A vapour barrier may be required depending on floor type.

3) Exterior finished grade away from wall at a minimum grade of 2% for min. 1.2m.

4) Building floor slab-on-grade shall not be structurally connected to foundation wall or footing.

5) Subfloor drain invert to be a minimum of 300mm below underside of floor slab, to be set in parallel rows, one way, and at the spacing specified in the geotechnical report. Don't connect subfloor drains to perimeter drains.

6) Embedded ports to be set a distance of maximum 3m on-centre. Each port to have a minimum cross-sectional area of 1500mm². Perimeter drainage must be collected and conveyed directly to the building sumps in solidpipe.

7) When the subgrade consists of a cohesionless soil, the subgrade must be separated from the subfloor drainage layer using a non-woven geotextile (Terrafix 360R or approved equivalent).

8) Geotechnical report contains specific details. Final detail must be reviewed before system is considered acceptable to use.

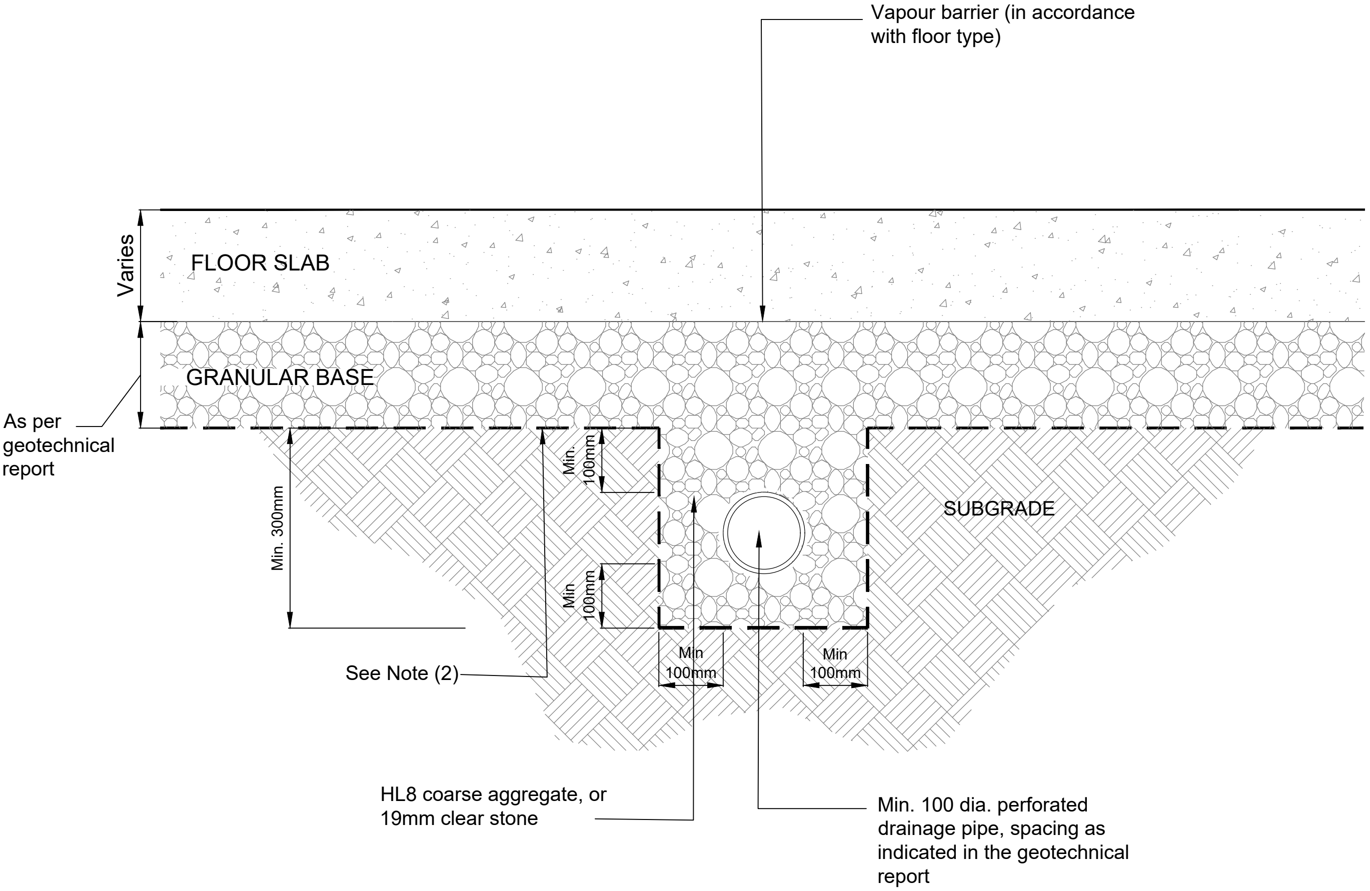
N.T.S.

Terraprobe

11 Indell Lane, Brampton, Ontario, L6T 3Y3
Tel: (905) 796-2650 Fax: (905) 796-2250

Title:

SCHEMATIC DRAINAGE DETAIL
SOLDIER PILE & LAGGING SHORING SYSTEM
(ONE-SIDED WALL CONSTRUCTION)



NOTES:

1. Typical schematic only. Must be read in conjunction with Geotechnical Report.
2. When the subgrade consists of cohesionless soil, it must be separated from the subfloor drainage layer using a non-woven geotextile (Terrafix 360R or approved equivalent).
3. Not to Scale

Terraprobe
11 Indell Lane, Brampton, Ontario, L6T 3Y3
Tel: (905) 796-2650 Fax: (905) 796-2250

Title:

TYPICAL BASEMENT SUBDRAIN DETAIL

APPENDIX E

TERRAPROBE INC.

PHOTOGRAPHS OF BEDROCK CORE SAMPLES	FIGURE B1

Proposed Redevelopment

Borehole No. 102

Borehole No. 104

Project No. : 3-20-0117
Date : April, 2021

Terraprobe Inc.

Prepared by : CM
Checked by : MS

Z:\1-Project Files\2020\Branch Folders\3-20-0117 - Collingwood Shipyards Block 6, Collingwood\01-1 - Geotechnical Investigation\C. Photos\Core Logs\3-20-0117\Rock Photos 1-19-0042-05 BH 201.xls

PHOTOGRAPHS OF BEDROCK CORE SAMPLES

FIGURE B2

Proposed Redevelopment

Borehole No. 108

Project No. : 3-20-0117

Date : April, 2021

Terraprobe Inc.

Prepared by : CM

Checked by : MS